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Proceedings

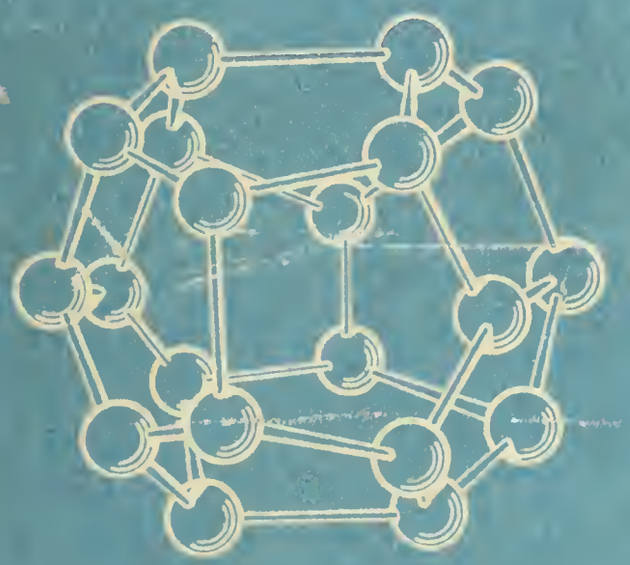
CONFERENCE ON WATER RESOURCES

October 1-3, 1951

HYDROLOGY

RADAR-WEATHER

CHEMISTRY



Dedicating

WATER RESOURCES BUILDING

Bulletin 41

ILLINOIS STATE WATER SURVEY

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STATE OF ILLINOIS
DEPARTMENT OF REGISTRATION AND EDUCATION

DIVISION OF THE
STATE WATER SURVEY
A. M. BUSWELL, Chief

BULLETIN NO. 41

PROCEEDINGS OF THE CONFERENCE ON WATER RESOURCES OCTOBER 1, 2, 3, 1951

ON THE OCCASION OF THE

DEDICATION OF THE WATER RESOURCES BUILDING

Printed by Authority of the State of Illinois

URBANA, ILLINOIS
1952

ORGANIZATION

STATE OF ILLINOIS

Hon. ADLAI E. STEVENSON, Governor

DEPARTMENT OF REGISTRATION AND EDUCATION

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A. M. BUSWELL, Ph. D., Chief

SCIENTIFIC AND TECHNICAL STAFF
of the
STATE WATER SURVEY DIVISION
October 1, 1951

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ACKNOWLEDGEMENTS

The organization of the program for the Conference and publication details were largely in the hands of Mr. H. E. Hudson, Jr. Associate Engineer W. J. Roberts was in charge of local arrangements and publicity. Associate Engineer Ross Hanson assembled the papers for this bulletin and arranged them for publication. Assistant Engineer J. B. Stall handled registration and Assistant Engineer R. E. Roberts took care of inspection trips. Professor R. K. Newton, Supervisor of Engineering Extension, University of Illinois, and his staff relieved the committee of many of the conference management details. Mrs. Lois McAllister was compositor for the entire publication and Mr. Stanley A. Changnon, Jr., did the drafting and design.

The staff of the State Water Survey wishes to express its appreciation for the invaluable assistance of the following:

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David Atlas, Project Scientist, Cambridge Air Force Research Center, Cambridge, Massachusetts.

A. C. Bemis, Project Supervisor, Weather-Radar Project, M.I.T., Cambridge, Massachusetts.

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Our gratitude is also due to those who have prepared the papers and discussions which contribute permanent value to the records of this occasion.

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DEDICATION OF THE WATER RESOURCES BUILDING

FOREWORD

The following are remarks made by Director Engle and Dean Larsen at the banquet dedicating the new Water Resources Building; this banquet was held Tuesday evening, October 2, 1951, in the grand ballroom of the Illini Memorial Union Building, Urbana.

PRESENTATION OF THE WATER RESOURCES BUILDING TO THE UNIVERSITY OF ILLINOIS

BY C. HOBART ENGLE*

It is indeed a pleasure for me to be here this evening and to bring you the greetings of Governor Stevenson. We in the State Administration recognize the fine and important work being done by the Water Survey. I shall never forget the time, as a youngster, I made my first visit to California. As we crossed from pleasant, green, vegetated areas into the barren desert, the value of water became obvious to me. In my present capacity I have come to appreciate more the great importance of water.

The Department of Registration and Education has received letter after letter of commendation. These come from communities, from cities, from industries and railroads, in appreciation for the assistance received from the Water Survey in water supply problems. I note one very interesting series of correspondence with a railroad which had asked the Water Survey for information on availability of water along its right-of-way and had received a comprehensive report pointing to locations where prospective industries might find water supplies to suit their needs. This is only one of the many cases in which the Water Survey serves the interests of the State of Illinois in the field of water resources.

The Water Resources Building which we are dedicating, cost slightly in excess of half a million dollars. Most of you present this evening have inspected this structure during this conference. I would like to commend Dr. Arthur M. Buswell, Chief, and the highly competent members of the Water Survey staff for their attention to the most minute details in the design and layout of this fine working headquarters.

This building was constructed with funds appropriated to the Department of Registration and Education. The Illinois civil administrative code requires that the functions and duties of the State Water Survey shall be exercised at the University of Illinois, in cooperation with the University staff, and in University buildings. In compliance with this code, I deliver to you, Dean Larsen, this key to the Water Resources Building, with the assurance that in these new quarters, the activities of the Water Survey, in cooperation with the University, will more effectively serve to solve the many important water problems in Illinois.

*Director, Department of Registration and Education, State of Illinois, Springfield, Illinois.

ACCEPTANCE OF THE WATER RESOURCES BUILDING BY THE UNIVERSITY OF ILLINOIS

BY HENNING LARSEN*

Acting on behalf of President Stoddard and the University, I am very pleased to accept, Mr. Engle, the wonderful new Water Resources Building that is now in full operation on the University campus.

As a professor of English, I have seldom realized the importance of water. We all take it for granted. And not till we meet with experts do we begin to realize the great extent of the problems and appreciate why the University and the State Water Survey Division should cooperate in the great effort of providing water for every purpose in this world. Once in a while even a professor of English realizes the need and importance of water. I happened to be in New York two years ago when we were told not to shave every day to save water. I had another practical experience some years ago when I was younger and climbed Long's Peak. Our guide on one occasion cautioned us, "Don't drink on the way up; wait for lunch on top." When we came to the top there was no snow, no puddle, and no drink; and then we found that a very imaginative cook had provided a lunch of hard-boiled eggs and peanut butter sandwiches! Even more, when later I sat for President Stoddard on the Board of Natural Resources, did I realize the whole of the problem.

We are proud to have Dr. Buswell, Water Survey Chief, as part of the College of Liberal Arts and Sciences in the Department of Chemistry. I know Dr. Buswell and his staff will continue to cooperate and that the University will give his organization its full support. The Water Resources Building is a creditable addition to our physical plant; on behalf of the University we accept it and say: Thank you very much for providing it.

*Dean, College of Liberal Arts and Sciences, University of Illinois, Urbana, Illinois.

EARLY DAYS AND ORGANIZATION OF THE ILLINOIS STATE WATER SURVEY

BY EDWARD BARTOW*

The Illinois State Water Survey was established in 1895 by the Trustees of the University of Illinois. It was the object to make chemical examinations of the waters of the State to determine the sanitary condition of any drinking water used by citizens of the State.

Professor Arthur W. Palmer was the Director. He was Head of the Department of Chemistry of the University and served until his death in February 1904. Professor S. W. Parr took charge of the work till September 1905.

Quarters and equipment were provided. Methods of chemical analysis were adopted and standards for the quality of water from the chemical analysis were formulated.

The response by the people of the State was immediate. Of 13,873 samples analyzed to January 1906, there were 5,376 sent in by private citizens or local health officers. An important piece of work was done for the Sanitary District of Chicago in 1899 and 1900 during the investigation of the effect of the Drainage Canal on the water of the Illinois River. There were 2800 samples analyzed. Special assistance was provided for the work. In addition to the regular staff, C. V. Millar and R. W. Stark, help was given by F. C. Koch (later Head of the Department of Biochemistry of the University of Chicago), A. D. Emmett (with Parke·Davis and Company), and A. L. Marsh (later General Manager of Hoskins Manufacturing Company).

Since my knowledge of the State Water Survey from 1895 to 1905 is by the reports, of which two were published by Professor Palmer, and by hearsay, it is proposed to consider the early days of the State Water Survey to include incidents that occurred from 1905 to 1920.

Professor Parr told the new Director that, in addition to the work of the Survey, it was hoped that there might be organized a course in Pharmaceutical Preparations. So much time was required by the Survey that the latter project never materi alized.

Prior to September 1905, a study of the quality of the water in a local stream had been begun. An old gentleman, a Mr. Snyder, was employed to collect samples. One day he asked the privilege of taking bottles to collect samples from his home well. A few days later he brought back the empty bottles with the statement that his wife would not allow the water to be analyzed because we would condemn it and then she could not drink it any more.

DR. ARTHUR W. PALMER
1895 - 1904

The funds for the support of the Survey were inadequate and cooperative agreements were made to supplement them. Bacteriological analyses were made with the help of the State Board of Health, which furnished a man to do the work, and equip-

*Director, Illinois State Water Survey, 1905-1920.

ment was supplied by the University. Dr. Walter G. Bain was appointed and remained with the Survey for one year. An ambitious program was planned to study epidemics, sewage purification, water supplies and streams. It was proposed to take over the operation of the Urbana septic tank and to try to make it an example for the operation of other tanks in the State. On the first visit to the Urbana tank, which was finished, we found that the sewage was being by-passed. We turned it into the tank.

DR. EDWARD BARTOW
1905 - 1920

Another cooperative agreement was between the United States Geological Survey, the Illinois State Water Survey, the Engineering Experiment Station of the University of Illinois, and the State Geological Survey of Illinois. $8200 was contributed for the work. Two projects were carried out. Daily samples of water were collected from 28 stations in the State and mailed in 4-ounce bottles to the Survey. These were composited into 3 lots per month and analyzed according to the methods of the United States Geological Survey. W. D. Collins of the Geological Survey was detailed to do the work. (He has recently retired as chief of the Division of Quality of Water of the U. S. Geological Survey.) The results of the work were published as Water Supply Paper 239 of the U. S. Geological Survey.

Summer field work in the East St. Louis quadrangle was done by Isaiah Bowman and Chester A. Reeds. Isaiah Bowman later held important positions, but is best known as President of Johns Hopkins University. A report of the results was published by the State Geological Survey.

Besides these two problems, mineral analyses were made of surface waters, groundwaters, and boiler waters, and sanitary analyses of surface waters and groundwaters. Analyses of the boiler waters were made under the direction of Professor Parr by F. K. Ovitz.

While cooperative work was worth while, it was hoped that the Survey could obtain sufficient funds from the Legislature to be independent, and the Director became a lobbyist—this with the approval of President Edmund J. James, who placed the responsibility in the hands of the Director. About this time legislation had been passed making some towns dry. A request for bottles for the collection of samples of water came from one town accompanied by the statement, "The town has gone dry and we do not have anything to drink."

At the session of the Legislature, we had the support of a newly elected member of the House, who had been mayor of the city for which the Survey had made analyses. He introduced a bill for additional appropriations for the Survey. He said he had been assured that the committee would pass the bill and that it would pass the House. It never got out of the committee. I think it was acted upon by the Senate.

At the next session of the Legislature, we thought we had the support of a prominent member, but he double-crossed us and the appropriation through the University was reduced. The Survey funds were saved by my assignment to teach Quantitative Analysis for a year.

The turning point in the affairs of the Survey occurred when a Conference of Water Works Officials and the Survey was held on February 15 and 17, 1909. Letters were read from President Edmund J. James and from Governor Charles S. Deneen, approving the objects of the Conference. Dean E. J. Townsend gave an Address of Welcome to which Dabney H. Maury of Peoria responded for the Water Works men. Papers were read, and a permanent organization was formed. Among about forty members present were representatives of municipal and privately owned water works, engineers, chemists, and representatives of supply houses. It is not pos-

sible to mention all of those present, but it should be noted that R. W. Bingham was registered from the Danville Water Company. At the time he was publisher of the Louisville Courier Journal and later was Ambassador to Great Britain. Charles Boeschenstein, Secretary, Vice President-Treasurer of the Edwardsville Water Company, was a prominent Democratic official, National Committeeman for Illinois. Mr. George J. Long registered as President of the Danville Water Company. He was President of a large steel company and a winner of the Kentucky Derby.

A committee of the Water Supply Association went to Springfield. The Legislature was nonpartisan in its dealing with the Water Survey. When the committee, including Mr. Henry Morgan, a Democrat, General Manager of the Peoria Water Company (Mr. Morgan had been postmaster of Peoria during both of Cleveland's administrations) walked on the floor of the House, Mr. Gorman, a Democrat, a leading member of the House and the representative from Peoria, greeted him with, "How are you, Henry? What can I do for you?" Mr. Morgan quietly said, "I am here in the interest of the Water Survey bill. I would like to see the Speaker." "Right this way," said Mr. Gorman, and we were all ushered into the office of the Republican Speaker.

The Legislature in 1911 made an appropriation so that we could employ engineers and establish an engineering division. The appropriation was $15,000. The previous appropriation had been $3,000.

When a visit was paid to the laboratory of the State Board of Health of Massachusetts at Lawrence, the Director stated that he employed high school graduates as they did not leave him. It was the policy of the Water Survey to train men to take positions elsewhere. One of our men became Director of the Lawrence Laboratory.

It is not possible to mention all of the men who have been connected with the Survey and to tell what they are doing or have done. I will refer you to a manuscript of an Autobiography that has tried to tell it in 45 pages. The following are some that are known to many of you:

The first M.S. was granted to Justa Lindgren. Then followed Lewis I. Birdsall, who has just retired from the headship of the Division for Water Treatment Chemicals of the General Chemical Company. Frank Bachman was Sanitary Engineer for the Dorr Company. W. F. Langelier is Associate Professor of Sanitary Engineering at the University of California. Fred Wilbur Tanner has been head of the Department of Bacteriology at the University of Illinois. Floyd William Mohlman is Chief Chemist of the Sanitary District of Chicago. Charles Herbert Spaulding was for many years in charge of the water purification plant at Springfield, Illinois. William D. Hatfield is in charge of the sewage purification plant at Decatur, Illinois. Friend Lee Mickle is Director of the Bureau of Laboratories of the State Board of Health of Connecticut. Robert E. Greenfield is General Manager of the A. E. Staley Manufacturing Company factory at Decatur, Illinois. Sidney D. Kirkpatrick is Consulting Editor for some of the McGraw-Hill publications and, I believe, a vice president of the company. Otto M. Smith is head of the Department of Chemistry and Chemical Engineering at the Oklahoma Agricultural and Mechanical College. C. C. Larson is in charge of the Water Purification and Sewage Disposal Plant of Springfield, Illinois. Gail P. Edwards, after holding various places in water and sewage purification, is now a professor in New York University. Robert C. Bardwell is Superintendent of Water Service of the Chesapeake and Ohio Railway. Eli Mandel is chemist for the Commonwealth Edison Company. William C. Marti is head chemist for the Chicago Department of Health, Carl J. Lauter is chief chemist of the Washington, D. C. filtration plant, and Arthur F. Mellon is superintendent of filtration at Minneapolis. We are proud of the part these men have taken in helping to obtain pure water for the country.

These and other members of the staff and graduate students have made possible not only the routine analyses, but many researches, that have been made the basis for theses, have been published in the 14 reports of the State Water Survey from 1905 to 1920.

These researches include studies of the use of bleaching powder for sterilizing water supplies, the activated sludge process (several papers), the removal of iron from the University and city supply, pollution of Lake Michigan by wastes from the Corn Products factory at Waukegan, the first experiments with Permutit in the U.S.A. It might also be mentioned that the Water Survey sent all its engineering staff to the Ohio River during the great flood of 1913. The Illinois Water Supply Association, founded under Survey sponsorship, became the first section of the American Water Works Association.

In conclusion, while I have many pages of manuscript in the Autobiography, I will close with the statement that the annual appropriation for the Water Survey increased from $3,000 to $35,000 from 1905 to 1920; it is nothing compared to what has happened since. I wish the Survey and its staff continued success.

THE STATE WATER SURVEY DIVISION
1920 -

BY A. M. BUSWELL*

To pick up the thread of the story of the Illinois Water Survey where Professor Bartow has left it, we need to refer to the reorganization of the State Administrative Organization accomplished by the Civil Administrative Code passed by the General Assembly in 1917.

The new law directed that the State Water Survey Division, cooperating with other Divisions, shall investigate and study the water resources of the State; prepare printed reports and furnish information fundamental to their conservation and development; cooperate with similar departments of other states and the Federal Government; study the geological formations of Illinois with reference to its resources in mineral and artesian waters; cooperate with the U. S. Geological Survey in the collecting, recording and printing of data on water resources, including stream-flow measurements; collect facts concerning the volume and flow of underground and surface waters of the State; publish from time to time, the results of its investigations of the mineral qualities, volumes, and flow of underground and surface waters, to the end that the available water resources of the State may be better known; make mineral analyses of samples of water from municipal or private sources and allied investigational and scientific research; cooperate with the University of Illinois in the use of scientific staff and equipment; and cooperate with the various departments in research, investigational and scientific work useful in the prosecution of the work of any department.

The absence of the Director and Chief Engineer in foreign service in World War I reduced the activities of the Survey somewhat, but on his return in 1919 Dr. Bartow started an expanded program in the study of the activated sludge process. This program was subsequently completed and resulted in the publication of several bulletins on both aerobic and anaerobic methods of sewage treatment. The explanation of the mechanism of the activated sludge process, the proof of the efficiency of the deep trickling filter and the application of anaerobic methods to industrial wastes were the principal contributions of these studies.

It was realized that, to more completely fulfill its responsibilities as outlined in the Civil Administrative Code, the State Water Survey Division should expand its activities in the direction of the study of water resources. By 1922 the engineering staff of the Water Survey had been detailed exclusively to the study of water resources, the first assignment being an inventory of municipal groundwater supplies. The results of this study were published in 1925 as Bulletin 21, comprising some 710 pages. During this study the late George C. Habermayer, then Chief Engineer of the Water Survey, introduced methods of determining water levels

DR. ARTHUR M. BUSWELL
1920 -

*Chief, State Water Survey Division, Urbana, Illinois.

in wells and emphasized the importance of well yield tests wherever possible. Following the inventory of groundwater supplies, a study of surface water supplies was undertaken. Various typical reservoirs were selected throughout the State, rain gages and staff gages were installed, and arrangements made to employ local gage readers to begin collecting data necessary for calculating the yield of a given watershed area.

At this time the cooperative stream-gaging program with the U. S. Geological Survey was provided for by a biennial appropriation of $6,000 running to the Illinois Division of Waterways. This appropriation was not considered adequate, and in 1933 the responsibility for cooperative work on stream-gaging was transferred to the State Water Survey Division and the biennial appropriation increased to $10,800. This appropriation has been repeatedly increased and at present amounts to $42,000.

The serious drought of 1936 resulted in a shortage of water supplies, both surface and ground, in many localities. This situation was aggravated by the tremendous industrial expansion during the early war years. To meet the increased need for study of water resources, the Legislature made an appropriation to the Water Survey of $100,000 at a special session called in the early spring of 1942. This appropriation made it possible to collect what has subsequently been found to be invaluable data on water recessions in important industrial areas and to point out methods for supplementing these supplies.

The chemical work of the Water Survey, which first began in the analytical laboratory and then passed to the experimental sewage treatment plant, has during the last 15 years shifted its emphasis to the question of quality of water for industrial uses and with particular emphasis on corrosion problems and newer water-softening methods. The regular analytical program has, of course, been continued.

During the period of World War II the chemical staff of the Water Survey devoted its entire time to cooperating with the University of Illinois and the Chemical Warfare Service in the study of the detection of chemical warfare agents in water and the development of methods for their removal. Two defense weapons were developed as a result of this program. These weapons were taken over by the Army and produced in quantity for use in case toxic chemicals were employed in warfare.

PRESENT ORGANIZATION

The Water Survey Division is now departmentalized into the following technical Subdivisions: Engineering, Engineering Research, Chemistry, Chemical Research.

The Engineering Subdivision is concerned with groundwater and surface water hydrology, the Champaign hydrologic laboratory, meteorology, municipal groundwater surveys, reservoir sedimentation.

The Engineering Research Subdivision is responsible for groundwater studies in the Peoria region and for the infiltration pit and laboratory located at Peoria, for groundwater studies in the Chicago-Joliet region, and at Cairo.

The Chemical Subdivision is concerned with chemical analysis of waters in Illinois and for research on the properties of water and on methods of treatment of the various waters for various uses. The correlation and quantitative interpretation of analytical data with respect to hydrology is an important activity of this subdivision.

Under Chemical Research the Water Survey is working in the field of supersonics; chemistry of water substance, in cooperation with the Navy and the University Department of Chemistry; and nitrification studies in cooperation with the U. S. Public Health Service.

ENGINEERING

GROUNDWATER SERVICES

Individuals and organizations in Illinois send many requests for groundwater information to the Water Survey. During the year 1950, there were 423 reports prepared in answer to these requests, many of which consisted of summaries of the existing data on groundwater developments and possibilities, prepared in cooperation with the State Geological Survey.

An average of 80 pumping tests of wells are made by staff members each year.

Water-level recorders are maintained in 85 observation wells, and data on water levels are recorded for some 1500 additional wells. This intensive groundwater data-collecting program topping the half-century's existence of the Water Survey, during which a very considerable library of basic data had been assembled, resulted in the publication in 1951 of Bulletin No. 40, comprising 1379 pages and entitled, "Public Ground-water Supplies in Illinois."

AREAL GROUNDWATER STUDIES

In May 1939 the Peoria Association of Commerce requested the Water Survey to make a study of the groundwater resources of the Peoria-Pekin area, the depletion of these resources, and the remedy for obtaining adequate water resources for the area. An intensive areal investigation of all city and industrial well supplies was made. Total groundwater use in the area is estimated to be 85

mgd. From this study it became evident that one of the best ways of replenishing Peoria's groundwater resources would be through the construction of an infiltration pit. As early as 1943 the Board of Natural Resources and Conservation passed a resolution which recognized that an emergency existed in the groundwater supply at Peoria and that an artificial recharging pit might be used to meet this emergency. On May 18, 1951, a permanent infiltration pit was placed in operation in the Peoria area. In 1949 the Water Survey staff for the Peoria-Pekin region moved into a new three-story laboratory building constructed in the city of Peoria near the shore of the Illinois River. The infiltration pit is constructed between the laboratory building and the river.

CHICAGO-JOLIET AREA

Principal source of groundwater in the Chicago area is in the deep sandstones. Total groundwater use in the area is 83 mgd. The Water Survey maintains three field engineers in the area collecting water levels, pumpage data, and other information on groundwater.

EAST ST. LOUIS AREA

Water levels and pumpage data from wells in the American Bottoms are recorded routinely by a field engineer stationed in the area. The groundwater levels show few changes; however, the wells close to the Mississippi River rise somewhat, corresponding to the effect of the high river stages. Groundwater use totals nearly 100 mgd. in this one area, which is one of the most intensively developed sources of groundwater in the world. Use is steadily increasing. Three high-capacity groundwater collectors have been installed in recent years.

SURFACE WATER

Industrial Use. A study has been made of all industrial and irrigation uses of surface water in Illinois. Of the 10 billion gallons per day used by industry, 0.85 billion gallons per day could be considered consumptive use and 9.15 billion gallons per day nonconsumptive use. This amount is divided up amongst (1) manufacturers - 7 billion gallons per day, (2) steam power - 2.8 billion gallons per day, and (3) mineral processing - 0.2 billion gallons per day.

Irrigation Use. There are approximately 200 irrigation systems in Illinois at present in five principal categories: truck gardens, flowers, pasture, forestry, and corn. This represents an investment of over $500,000, a potential capacity of 100,000 gallons per minute, and a total irrigated area of 9,000 acres. The water resources in use are primarily surface, with groundwater providing about 20 percent of the amount pumped.

STREAM GAGING

Stream-gaging activities are carried on under the usual cooperative arrangement with the Water Resources Branch of the U. S. Geological Survey in which costs are shared equally. There are now 156 stream-gaging stations in Illinois. Study of the runoff characteristics of Illinois watersheds using the stream-gaging data is carried on in the Water Survey offices. Correlated with this study is a study of stream discharge relations, the physical characteristics of the basins, surface retention and detention and overland flow, infiltration rates, and hydrograph analyses.

For nearly 30 years the Survey has maintained a research project on small watersheds in Illinois, which includes compilation of measurements of precipitation, discharge, storage, and pumpage of the five water supply reservoirs in Illinois.

RESERVOIR SEDIMENTATION

At the present time, sedimentation accumulation data are available on 41 reservoirs in Illinois. Most of this information has been collected during the last 10 years under the cooperative research agreement between the State Water Survey, the Soil Conservation Service, and the Illinois Agricultural Experiment Station. Detailed sedimentation surveys have been made on 18 reservoirs, and reconnaissance surveys have been made on 17 reservoirs. Cross-sections have been taken on six new reservoirs during construction.

This program is furnishing results of value to waterworks officials who are responsible for the dependability of impounding reservoirs. In the field of agriculture this program is furnishing data on sediment production from known soils under various conditions.

METEOROLOGY

Of all hydrologic data, precipitation measurements have proved least trustworthy; however, the Water Survey's present program has begun to produce data of much improved dependability. The principal phases of the program are: (1) the El Paso areal rainfall study, (2) the Goose Creek areal rainfall study, (3) Boneyard Creek hydrologic study (in cooperation with the Department of Civil Engineering, University of Illinois), (4) gaging installations in conjunction with other stream flow and groundwater studies, (5) auxiliary radar installations, and (6) the University Weather Station. The program involves approximately 60 automatic rain

gages. Results of the studies of rainfall measurement with radar have produced a quantitative relationship between radar signal strength and rainfall intensity.

The Weather Station makes approximately 2,000 reports per year. Of these, 270 are to the U. S. Weather Bureau; 600 to University departments; to other organizations and individuals—1130.

The Water Survey maintains three Class "A" Weather Bureau-type pan evaporation stations in Illinois located at Carbondale, Urbana, and Rockford. In addition, there is the U. S. Weather Bureau station at Springfield, Illinois. Data collected at the Water Survey stations are published by the U. S. Weather Bureau. The Water Survey maintains a study of evaporation by determining vapor-pressure gradients through sensitive dewpoint recorders at Four-Mile Crib located in Lake Michigan and at Urbana (in cooperation with the University of Illinois Department of Civil Engineering).

CHEMISTRY

ANALYTICAL METHODS AND ANALYSES

The need for new or especially modified methods for testing water has continued since the earliest days of the organization. Recent contributions include methods for the determination of methane gas, sodium, hardness, control of sewage treatment, and feed water for power plants.

Circular No. 31 gives complete chemical analyses of all public groundwater supplies in Illinois. This circular, which includes a discussion on the significance of mineral ingredients in water, is the result of frequent requests and considerable interest in the chemical quality of public water supplies aside from the physical data on the source of water.

The following annual statistics on analyses indicate the magnitude of the Water Survey laboratory work: bacteriological 1,856; mineral 184; partial mineral 596; gas 6; special chemical tests 812. This gives a total of 3,454 chemical analyses in an average year.

CORROSION

A survey of the corrosive characteristics of Illinois water has made possible recommendations as to suitable materials for pumps, valves and other equipment. A large number of requests for such information are received regularly. The hydraulic laboratory offers opportunity to test corrosive action under operating conditions.

WATER FOR STEAM POWER PLANTS

Recently the Water Survey has been assigned the control of feed water treatment for the power plants at State institutions. Besides resulting substantial financial savings and improved operation, this work is developing information of value to many private industries.

Testing of New Techniques. The evaluation of new techniques in the solution of the problems of water chemistry is a continuing activity. The use of infrared spectroscopy, supersonics, exact specific gravities, solubilities, tracer isotopes and the structure of water substance, are recent and current topics under investigation.

PUBLICATIONS

The State Water Survey Division and its predecessor, Illinois State Water Survey, has published 40 Bulletins, 35 Circulars, and 15 Reports of Investigations.

WATER RESOURCES CONFERENCE

On October 1-3, 1951, a Water Resources Conference was held on the occasion of the dedication of the new Water Resources Building, housing the offices and laboratory of the State Water Survey Division, on the University of Illinois campus at Urbana.

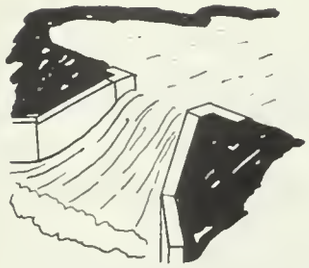
There were more than 475 registrations for the three concurrent programs of HYDROLOGY, TREATMENT, and RADAR-WEATHER. Nearly all of the registrants were from outside of Champaign-Urbana, and a large proportion were from outside the State of Illinois.

The sessions for Hydrology and Treatment were held in the Illini Union and East Chemistry Buildings, respectively, located on the University of Illinois campus. The Radar-Weather sessions were held on the Allerton Estate 20 miles southwest of Champaign-Urbana, where all meetings, housing and meals were provided under one roof.

The very large attendance was very gratifying and may be attributed in part to the very high quality of the conference program and, to a large extent, to the careful planning and preliminary publicity. All members of the staff contributed unstintingly to the extra effort required to make the conference a success. The burden fell heaviest on Mr. H. E. Hudson, Jr., Mr. W. J. Roberts, Mr. Glenn E. Stout, and Dr. T. E. Larson.

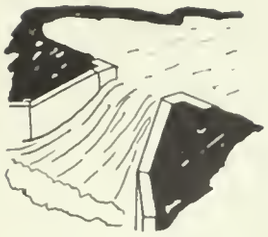
Seventy speakers took part. All of the papers presented at each of the sessions were on topics of immediate interest and were warmly received, as indicated by both the prepared and impromptu discussions. The State Water Survey Division is pleased to present this Bulletin No. 41, which contains the papers and prepared discussions as presented, together with a transcript of the oral discussions, with the hope that it will be a valuable reference work for all in the field of Water Resources.

HYDROLOGY



Program Chairman

HERBERT E. HUDSON, JR.



Hydrology

Technical sessions to be held in Room 314, Illini Union Building.

MONDAY, OCTOBER 1

Morning

Registration at Water Resources Building, 605 East Springfield Avenue, Champaign

* * *

Afternoon Session
Wallace M. Lansford, Presiding

2:00 "Hydrology and the Hydraulic Laboratory," Albert S. Fry
2:30 Discussion: Arno T. Lenz, J. I. Perrey, Carl E. Kindsvater
3:00 Discussion from the floor

3:15 "Bed Load Function for Sediment Transportation in Open Channel Flows," Hans A. Einstein
3:50 Discussion: Vito Vanoni, Ralph W. Powell
4:15 Discussion from the floor
4:30 Adjournment

* * *

Evening

6:30 Informal get-together, Urbana-Lincoln Hotel

Buffet supper.
Football and radar movies.

Hydrology

TUESDAY, OCTOBER 2

Morning Session
Alex Van Praag, Presiding

9:30 Panel discussion: "Sedimentation Problems" L. C. Gottschalk, Moderator
9:35 "Basic Data Collection," Gunnar Brune
9:50 "Application of Sedimentation Data to Water Project Design," N. T. Veatch
10:05 "Watershed Approach to Sedimentation Problems," O. W. Chinn, Wendell LaDue
10:35 Discussion from the floor

10:45 "Analysis and Use of Groundwater Data," William F. Guyton
11:20 Discussion: John H. Bliss, Howard Critchlow, H. A. Spafford
11:50 Discussion from the floor

Afternoon Session
J. J. Doland, Presiding

2:00 "Analysis and Use of Surface Water Data," Richard Hazen
2:35 Discussion: E. F. Brater, W. D. Mitchell, C. V. Youngquist
3:00 Discussion from the floor

3:15 "Engineering Meteorology," Stifel W. Jens
3:55 Discussion: George Benton, Ivan E. Houk, Phillip Light
4:20 Discussion from the floor

* * *

5:30 Reception, Music Lounge, Second Floor, Illini Union Building
6:30 Banquet, Grand Ballroom, Illini Union
The program will include the dedication ceremony, and an address by Clark M. Eichelberger, National Director, American Association for the United Nations.

Hydrology

WEDNESDAY, OCTOBER 3

Morning Session
Horace Gray, Presiding

9:30 Panel discussion: "Water Use"
R. G. Snider, Moderator
"Industry's Water Problems," T. J. Powers
"The National Picture," Walter Picton
"Health Aspects," Paul W. Reed
"Industrial Conservation Methods,"
Howard E. Degler

10:30 Discussion from the floor

10:45 "State Water Policies," Louis R. Howson
11:25 Discussion: R. O. Joslyn, Thorndike Saville
11:50 Discussion from the floor

Afternoon Session

2:00-5:00 Conference of State Water Resources Agencies

This session includes the reporting of results of a questionnaire sent to all State Water Agencies in the Nation on collection of basic data. These data will be summarized at the meeting. Discussion of administrative, fiscal, and other state problems is being planned by a committee consisting of C. V. Youngquist, Charles Bechert, Howard Critchlow, T. E. Larson, Earl Sanderson, H. E. Hudson, Jr.

HYDROLOGY AND THE HYDRAULIC LABORATORY

BY ALBERT S. FRY*

WITH DISCUSSIONS BY ARNO T. LENZ, J. I. PERREY AND C. E. KINDSVATER

THE PLACE OF HYDROLOGY

The period since the early 1930's has been one of great activity in the development of the water resources of the country. Many single-purpose and many multiple-purpose projects have been built in the interests of flood control, navigation, hydropower generation, irrigation, water supply, and other purposes. Many more such projects are contemplated for the future. In the planning, design, and operation of such projects, the science of hydrology has an essential and important part. It is logical then that, along with the increased water project development, the science of hydrology itself should have gone forward at an accelerated pace so that today hydrology, though specialized to a considerable degree, is recognized as being a science of major importance.

As defined in a recent book, Applied Hydrology by Linsley, Kohler, and Paulhus, hydrology is "that branch of physical geography dealing with the waters of the earth with special reference to properties, phenomena, and distribution. It treats specifically of the occurrence of water on the earth, the description of the earth with respect to water, the physical effects of water on the earth, and the relation of water to life on the earth." According to the American Geophysical Union it "is a borderline science of interest to agronomists, engineers, foresters, meteorologists, soils technicians, geologists, and others." The fields of interest according to the AGU are precipitation, runoff, infiltration and percolation, evaporation and transpiration, chemistry of natural waters, ground water, soil-moisture, dynamics of streams, including transportation and deposition of sediment and movement of flood-waves, limnology, glaciers, and snow and snow surveying.

MODERN HYDRAULIC LABORATORY

Where does a hydraulic laboratory fit into such a framework? Before discussing this specifically, it is appropriate to consider what constitutes a modern hydraulic laboratory. Broken down into its simplest and most basic components the hydraulic laboratory must consist of five elements: space, a water supply system, a collection of measuring instruments, shop facilities, and a staff. While it is difficult to select any one of these five as more important than the others, probably an adequate staff is the most important of all.

Present day hydraulic problems are in general so complex that a staff with a very broad training is required. At the present time most laboratory staffs consist not only of civil engineers trained in the field of hydraulics but also of electrical engineers, mechanical engineers, chemical engineers or chemists, and there is a growing tendency to incorporate one or more physicists. Because of the extreme diversity of the problems encountered, each of the staff members must have a broad knowledge in his specific field rather than be a specialist in any one phase of that branch. The laboratory staff must include competent craftsmen who can handle almost any type of small construction that may arise. Since much of the work performed in a laboratory cannot be done with purchasable items, these craftsmen must be capable of fabricating almost any type of article that might be required in such a laboratory. The item may be a complicated electrical instrument or a large concrete weir. It may be a relatively simple piece of sheet metal or a complex machined item.

THE LABORATORY IN HYDROLOGY

With such a staff and the other essential components of a hydraulic laboratory, the laboratory is properly equipped to make significant contributions to many and varied phases of hydrology.

The areas in which the laboratory may be of primary value include (1) the development of special devices and equipment, (2) ratings and calibrations, (3) manufacturing special devices, (4) maintenance of hydrologic equipment, and (5) working out solutions for hydrologic problems, either in the realm of fundamental research or specific problems encountered in a particular hydrologic project.

DEVELOPMENT OF SPECIAL DEVICES AND EQUIPMENT

Further advancement in several important hydrologic fields waits upon either refinement of available instruments or development of new ones.

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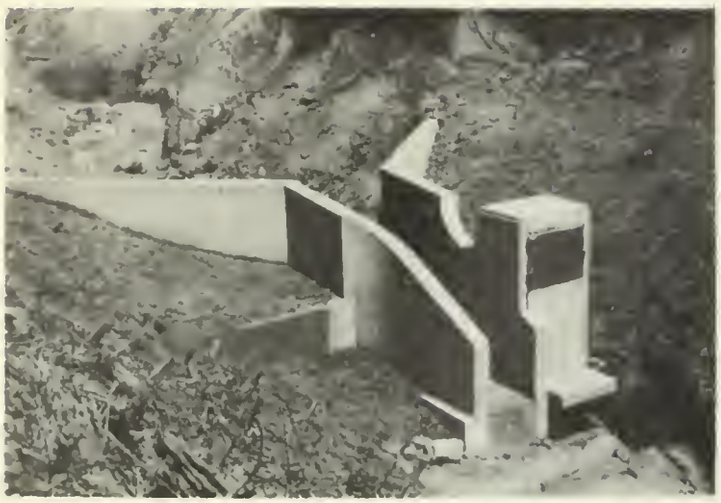

FIG. 1. —SAN DIMAS FLUME.
Developed by the U. S. Forest Service, this flume is adapted to the measurement of small stream discharge. Particularly suited for streams with heavy sediment loads, this flume is a useful tool in hydrologic studies of small stream runoff.

FIG. 2. —H-FLUME FOR SMALL STREAMS.
This measuring device, developed by the Soil Conservation Service by laboratory methods, is useful in watershed hydrologic investigations.

The failure to solve some of hydrology's major problems is due to the lack of the proper instruments for making the measurements required to obtain the solutions. The hydraulic laboratory can definitely be of service in the matter of improved instrumentation. Some instruments developed for laboratory use may be useful in hydrology without change since many hydrologic problems are closely akin to those encountered in hydraulic laboratory practice. In other cases, specialized instruments will need to be designed, built, and calibrated by the laboratory for use specifically in the hydrologic field. Since the facilities and staff requisite for development of instruments and special devices are required for the laboratory, their further use on hydrologic developmental work is a logical sequence.

An outstanding example of the collaboration of the hydraulic laboratory with hydrology in development of equipment, is the cooperative project established several years ago by six governmental agencies concerned with sediment loads in streams. Prior to that time, there had been no standardization of sediment samplers or methods of measurement and analysis of sediment loads in streams. The project was first set up at the Iowa Institute of Hydraulic Research and later moved to the St. Anthony Falls hydraulic laboratory. As a result of this project, sediment samplers were developed which have been generally accepted and standardization according to acceptable proven methods has brought order out of chaos in this type of hydrologic work.

In setting up watershed hydrologic studies some years ago, hydrologists needed flow measuring devices of greater sensitivity for low flow than could be obtained with ordinary types of weirs. Accordingly, in a number of hydraulic laboratories around the country special flumes and weirs were developed to meet the conditions encountered. Among these may be noted the San Dimas flume developed by the Forest Service, Figure 1, the H-flume developed by the Soil Conservation Service, Figure 2, and the Columbus deep-notch, Figure 7, and associated types of weirs developed at the Bureau of Standards for the U. S. Geological Survey. These examples typify rated measuring devices that have been developed by hydraulic laboratories for use in watershed hydrologic studies.

In the White Hollow watershed project in the Tennessee Valley, the problem of automatic sampling of suspended sediment was solved by the development of an automatic sampler in the TVA hydraulic laboratory. For a watershed of 1800 acres there was need to determine, accurately, changes in sediment from the area as natural revegetation and reforestation became progressively more effective. In the early years of the study an attempt was made to have suspended sediment samples taken at the control weir by a nearby farmer paid for taking samples according to a predetermined schedule. However, this method proved unsatisfactory as it is certain to in any similar case. When storms occurred at night, the creek came up and went down before next morning, but the farmer observer could not be depended upon to get up and go out into the night to take samples. To maintain satisfactory sampling, it was apparent that a dependable automatic sampler would have to be devised. The watershed is large for automatic sampling and it was desired that samples be taken for flows from low water up to a maximum of about 250 cubic feet per second.

FIG. 3. —AUTOMATIC SEDIMENT SAMPLER IN LABORATORY.

This device, developed and manufactured in the Hydraulic Laboratory, retains 1/105,000 of the total stream flow for sediment analysis..

It was decided that a sampler needed to be developed which would sample about 1/100,000th of the flow continuously. Such a sampler was worked out in the TVA hydraulic laboratory and was built there for installation in the field after having been completely rated. Figure 3 shows the sampler in operation in the laboratory. The development of a sampler to take such a small part of total flow is illustrative of the techniques and skills that the hydraulic laboratory can bring to bear on problems of this kind. At the sampler intake a weir plate, Figure 4, was developed that would draw from the stream an amount of flow accurately calibrated to be 1/100 of the flow. Beyond the weir plate, flow passes through a corrugated baffle and is then divided again with a tenth of the flow being retained. A 1/2 division of the flow was next made. This was followed by finally retaining only 1/50th of the remaining flow through the use of a tipping bucket mechanism at the lower end of the sampler. Actually the devise as finally calibrated takes 1/105,000th of the total flow. Figure 5 shows the sampler installed and in operation in the field.

One phase of hydrology is concerned with the dynamics of streams which involves wave travel and water travel in streams and lakes. In this connection, equipment was developed and rated by the TVA hydraulic laboratory for discharging fluorescein dye into natural streams over long periods of time at constant predetermined rates. In one case it was desired to color 80,000 acre-feet of inflow into a large reservoir and trace this through the reservoir. Another use of the equipment has been the dyeing of inflow into a lake used for atomic waste disposal to determine the time of water travel through the lake under unstratified and stratified thermal conditions. For use in streams polluted by industrial wastes, equipment has been designed and fabricated for measuring conductivity of the stream over a period of time at successive stations from which the time of water travel can be determined.

In TVA's hydraulic laboratory, the Instruments Unit is responsible for the design and con-

FIG. 4. —SENSITIVE WEIR PLATE DEVELOPED IN LABORATORY.

This plate was designed and carefully calibrated to pass 1/100 of the total flow of the stream on which it was installed.

FIG. 5. —SEDIMENT SAMPLER INSTALLED IN FIELD.

This is the sampler shown in Fig. 3 with the addition of metal covers and the collecting tank shown on the left. The entire sampler was manufactured by the Hydraulic Laboratory.

struction of radio rain and stream gages. The gages used by TVA have been developed by the laboratory technicians. At the present time, in order to meet requirements of the Federal Communications Commission, the laboratory is developing a new type of gage in the VHF band which is assigned for transmission of hydrologic data.

TVA's laboratory is currently working on an instrument designed to make use of a radioactive source in conjunction with suitable detection equipment to determine the density of deposited sediment.

In recent investigations in the Tennessee Valley having to do with atmospheric pollution, it was desired to obtain temperatures at ground levels and at heights of 200 to 300 feet in order to evaluate lapse rates and isolate inversions. Humidity was also desired. The TVA hydraulic laboratory using commercially available instruments designed to record differential temperatures developed the necessary auxiliary equipment and sensing elements to permit the observations to be made. Mounting for wind direction and velocity instruments together with housing for all of the instruments were designed by the laboratory as a part of the total installation. Figure 6 shows a typical installation of this kind.

RATINGS AND CALIBRATIONS

The second important function of a hydraulic laboratory is in the matter of rating and calibration of measuring devices and instruments.

In watershed hydrology on areas that may range from a few acres to several hundred acres, precise and continuous measurements must be made of stream flow. Because the physical areas are relatively small, stream flow peaks occur so quickly after precipitation that only occasionally is there time for an engineer to travel to the stream to measure the flow by current meters. Furthermore, unless the engineer is located very near the stream, he will not be aware of rises on the stream due to small area storms. Hence provision must be made for automatic determination of flow to be made by measuring devices for which ratings are available. In some cases standard measuring devices can be so installed as to provide for measuring stream flow without special calibration but more often than not adaptations will need to be made that require the facilities of a hydraulic laboratory for high flow calibration.

FIG. 6. —WIND AND TEMPERATURE OBSERVATION TOWER.

Instruments to determine wind velocities and directions, temperature gradients and humidity are mounted on this tower to furnish data needed in air pollution studies.

In most installations, low flow conditions will be the same so that the standard rating for measuring devices such as the Columbus deep-notch weir can be used. However, ratings for high flows are likely to vary from one installation to another due to differences in physical conditions encountered at different measuring stations. In the Tennessee Valley, it has been valuable in providing a rating for the measuring device above low flow to build the installation in the hydraulic laboratory either at natural or a model scale to calibrate the installation to the expected high flow stages. The model built in the laboratory takes account of approach conditions as well as the configuration of the abutments and sides of the channel. A rating when developed then represents conditions as they are in the field at the particular installation.

In one hydrologic study made by TVA, it was found that the rating for a Columbus deep-notch weir was materially affected by the deposition of bed load and heavy sediment in a settling basin immediately upstream from the weir. The condition might have been corrected in the original installation but due to limitations of available funds and other conditions this had not been done. The deposition of sediment introduced problems of velocity and direction of flow through the basin which affected discharge. Flows had to be computed under varying conditions of silt deposition, which fortunately were known as a result of rather frequent cross sections of the basin. The problem was taken to the hydraulic laboratory and a 1:5 scale model of the installation was constructed, including the measuring device and the sediment basin. Figure 7 shows the weir in the field and the model built in the laboratory to determine discharge rating. Tests were made to rate the weir for various conditions of past silting which made possible the reduction of records of stream stage into discharge. This illustration shows one way in which the hydraulic laboratory can be useful in reproducing field conditions and developing flow ratings under conditions which make it essential for ratings to be obtained by model studies.

Groundwater problems may find the hydraulic laboratory a valuable aid in calibrations and closely related work. These include determinations of permeability of various soils by laboratory methods. Particle size and volume weight determinations may be made of soil samples. Macropore space measurements and percolation rates may be run on undisturbed soil samples. Calibration of soil moisture units may also be assigned to the hydraulic laboratory. Soil moisture studies are achieving new and important significance in hydrology as the result of the availability in recent years of the Colman soil moisture fiberglas unit and the Bouyoucos nylon unit. Installation of these units in stacks ranging from the surface down to the lower limit of the top of the groundwater table provides a source of valuable information on soil moisture and soil moisture movement that has not otherwise been available. The cells require rather laborious ratings and checked determinations on field samples which can well be carried out by the hydraulic laboratory.

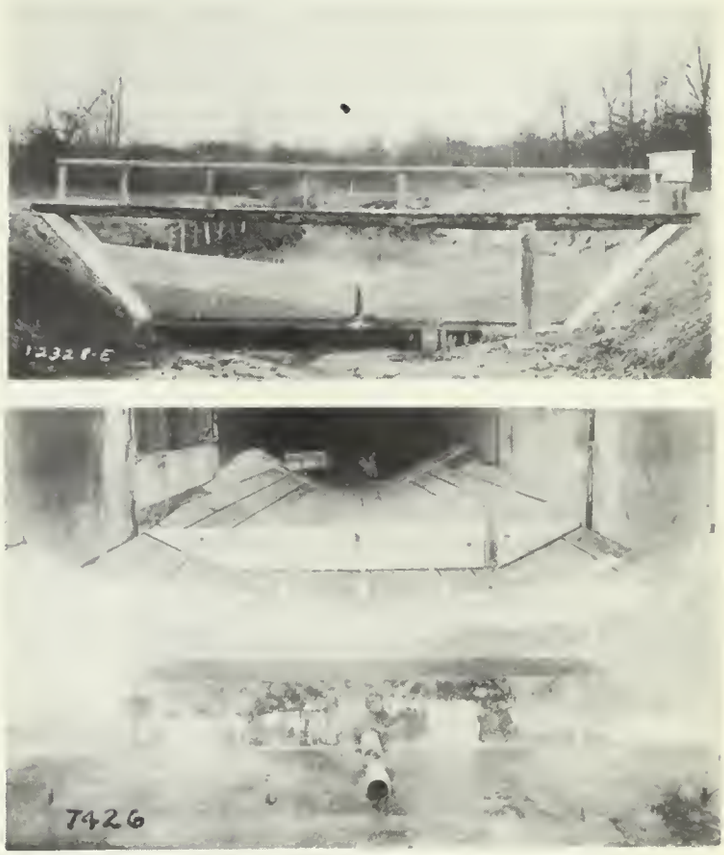

FIG. 7.
Upper view shows prototype field installation for which discharge rating was developed in Hydraulic Laboratory for various conditions of sediment deposition in basin above weir by the 1:5 scale model shown in the lower view.

Other examples of calibrations that have been made by TVA's hydraulic laboratory include calibrating electrical conductivity meters used in water travel studies and preparation of color standards for fluorescein dye concentrations also used in water travel investigations.

MANUFACTURE OF EQUIPMENT

Since shop facilities with competent craftsmen are a part of the hydraulic laboratory, it is logical that the laboratory manufacture many of the hydrologic measuring devices, special equipment and instruments. Much of this is special equipment not

available commercially. The shop facilities of the laboratory will ordinarily be adequate for such manufacturing since the machines available should be those originally selected for their versatility for doing nearly every type of normal construction and fabrication process to a greater or lesser extent. Examples of equipment built by TVA's laboratory facilities include continuous sediment samplers, measuring weirs, special soil samplers, conductivity meters, special instrument housing, sensing elements for temperature recorders, dye-dosing equipment, and radio precipitation and stream gages.

MAINTENANCE OF EQUIPMENT IN FIELD

Hydrologic equipment necessarily requires maintenance, the amount depending on the particular piece of equipment. Some of this maintenance is of a routine character that may be handled by relatively unskilled personnel but there is usually some maintenance of a higher order. Personnel from the hydraulic laboratory are well qualified for this, particularly in the case of equipment made by the laboratory. TVA laboratory personnel maintains an extensive system of radio rain and stream gages, special meteorological instruments, various electrical meters, and related equipment.

SOLVING HYDROLOGIC PROBLEMS

Many hydrologic problems that are particularly difficult or not otherwise susceptible of solution may be worked out in the hydraulic laboratory either at full or model scales.

A noteworthy illustration of this is the gigantic model of the entire Mississippi River with all its tributaries and reservoirs, parts of which have been constructed by the Corps of Engineers near Jackson, Mississippi. Covering a ground area of about 220 acres, this model is designed and instrumented to study flood flows throughout the Mississippi River and its principal tributaries.

In contrast with this huge model are such problems as the design of small soil conservation structures so that these will operate with minimum tendency to erosion at the outlets. These may be either standard designs or special designs for definite locations. Experimental model studies of this kind have been made at the St. Anthony Falls hydraulic laboratory.

In problems concerned with the dynamics of streams the hydraulic laboratory is a valuable aid to field investigations. In several laboratories throughout the country basic laws regarding transportation and deposition of sediment are being studied. On the Watauga River in the Tennessee Valley, the effect of intermittent discharges from a powerhouse was significant in questions of stream sanitation. Field observations were made of the phenomena occurring but a clearer understanding of wave travel and water travel in the stream was obtained after constructing and operating a model in the laboratory. Laboratory studies of density current behavior have contributed valuable knowledge supplementary to full scale field observations.

Determination of roughness coefficients for natural streams by field measurements and estimation of flood flows by slope-area and contracted opening methods all offer an opportunity for fruitful collaboration between the hydraulic laboratory and stream flow engineers.

Some problems in hydrology by their very nature may be solved only by full scale field studies but in some cases certain basic factors in the problems can best be studied when isolated in a laboratory. For example, in the field of evaporation and transpiration it is possible in the hydraulic laboratory to isolate factors in order to obtain basic information relative to the processes taking place. Dr. G. H. Hickox made an important contribution in the field of evaporation several years ago through experiments made in still air under controlled conditions at the University of California. Recent investigations on Lake Hefner in Oklahoma by the Navy, Weather Bureau, and Geological Survey are in fact hydraulic laboratory investigations carried on in the field under conditions where all variables can be precisely measured by research techniques.

With respect to sedimentation perhaps the hydraulic laboratory should not be considered an appropriate place for determination of suspended sediment loads and size analysis of sediment samples. However, it may prove under certain organizational setups that the laboratory can do work of this kind and certainly there is no limitation on the ability of the laboratory to carry out work of this type. A recent bulletin from California Institute of Technology describes laboratory tests that were made in studying the use of pervious fence for stream bank revetment and stabilization.

There has been a growing consciousness in recent years of the inseparability of land and water. The problems of one are the problems of the other. The type of cover on the land influences runoff distribution and yield as well as erosion of the soil. The fullest use, control, and conservation for the future of both the water and the land resources demand that the objectives for both be planned on a sound hydrologic basis. In providing this, the hydraulic laboratory because of its inherent relation to water has an important contribution to make.

USE OF HYDRAULIC LABORATORY STAFF IN HYDROLOGY

Many of the aptitudes and skills required are the same for hydraulic laboratory and hydrologic personnel. Both require people of high competence

who are at home in basic and applied research and the solution of complex hydraulic problems. The staff of the hydraulic laboratory must collectively have a comprehensive knowledge of a wide variety of engineering and scientific fields. Because of this fact and because so much of the laboratory work closely parallels various phases of hydrology, the laboratory technical staff can both contribute directly to many hydrologic projects and serve as a valuable consulting source to the hydrologists.

THE LABORATORY IN FUTURE STATE DEVELOPMENT

Throughout the state of Illinois, there are numerous problems of hydrology that need to be studied and solved as an integral part of the continued development of the resources of this great state. The scope of these problems embraces the entire field of water control, conservation and utilization. In connection with surface and underground water supplies, flood control, navigation, watershed protection, fish and wildlife, stream pollution, and recreational use of water, hydrology will play a large part in future developments in Illinois. Projects concerned with some of these activities will range from those of limited local import to large comprehensive developments affecting considerable areas of the state. In the hydrologic as well as other phases of these future programs, the new hydraulic laboratory of the Illinois State Water Survey with its modern facilities is certain to be a great asset with its sphere of influence stretching out to include the entire state of Illinois.

DISCUSSION

ARNO T. LENZ.*—The opportunity to discuss this paper is indeed a privilege and an opportunity: A privilege because there are many who could cite from vast experience a multitude of instances where the hydraulic laboratory has been an effective tool in increasing the store of hydrologic knowledge and its application, and an opportunity because there is so much need for the use of all old tools and many new ones in a field of study so relatively young in engineering.

The paper treats of many problems where the laboratory is of material aid in their solution. Of particular interest to me is the reference to the White Hollow silt sampler because (1) some 10 years ago I was briefly associated with that experimental area and (2) I am of the opinion that much light can be shed on many hydrologic problems by the study of short time variations in measurements, i.e.: rates of change, as contrasted to long time average results. The use of continuous recorders for measurement of streamflow, precipitation, and groundwater levels yields so much more information that the added expense is well justified in many instances. "Find out what happens" is a good adage and an instantaneous reading taken once in a 1 to 30-day period, or an average result over that period, will seldom give the desired information, particularly when studying small drainage basins. Obviously the hydraulic laboratory must take leadership in the development of the instruments required. Laboratories such as the one we are here to dedicate can and will do much to lead in these developments.

When the number of such special instruments required is small, their manufacture for sale or lease to various governmental and private agencies who have need for them should be a proper function of the laboratory, a function fraught with many perils as many well know.

The rating and calibration of measuring devices is indeed a function of the laboratory. In many instances model tests of existing structures are essential if the structures are to be used as measuring devices. However, lest there be a tendency to engage in wholesale calibration it may be well to emphasize the desirability of using the various weirs and flumes mentioned by the author rather than building "different" designs and calibrating them. The laboratory is a more economical tool when used for developing standard designs than when used for custom calibration.

The author mentions that many particularly difficult problems may be worked out either with model or full scale tests. One great advantage of laboratory work is the ability to isolate variables not readily isolated in the field. Since so many hydrologic problems are exceedingly complex this advantage is a very real one which is not recognized as much as it should be.

One function of laboratories, too often neglected in the press of daily demands for experimental results, is the responsibility of making available to all, through publications, the vast store of information already available. Engineers by nature are perfectionists. We can always see one more check test that would be desirable before we put our names on a report in print. Yet if we are willing to use the information in our own daily work, others also would be willing to make the same use of it. Writing papers is hard work and takes time but laboratory administrators should make that time available and encourage publication of results. Tax-supported laboratories particularly have a real responsibility in this regard.

Finally, as a teacher of engineers, it was very gratifying to read that the staff is one of the basic elements of the laboratory, probably the most basic of all. With that statement I am in hearty agreement. With ever-increasing use of our water resources dissemination of knowledge of their limits and methods of control is a responsibility the engineering profession must not pass by. Yet from a questionnaire prepared by the Committee on Hydrology of the Hydraulics Division of the American Society of Civil Engineers and reported on by me at the division meeting in Jackson, Miss. in November, 1950, it was evident that half the engineering schools in this country give no work in hydrology, other than that 10 to 30 per cent of such courses as Water Supply, Sewerage, etc. as should normally be used to show specific applications in hydrology to the particular field being studied. In that paper it was recommended that every college and university require that each civil engineering student take as a minimum a 2 semester-hour or 3 quarter-hour course in Hydrology before graduation.

The need for broad training of the laboratory staff is well considered by the author. That same broad training for all engineers is essential for proper functioning of the engineering profession and the national good. Civil engineers, at least, should be in a position to recognize hydrologic problems and support those with specialized knowledge in the orderly economical solution of those problems. Unfortunately, I feel this sermon isn't reaching the people who really need it.

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The staff of the Illinois State Water Survey Division are to be commended and congratulated on this fine Water Resources Building. Through its use in the solution of hydraulic and hydrologic problems they will contribute much to the state and national well-being.

J. I. PERREY.*—This meeting and Mr. Fry's paper on "Hydrology and the Hydraulic Laboratory" are both especially timely, because the science of hydrology is just coming of age. Prior to 1930, hydrology was considered to relate principally to underground waters, and the present concept that it embraces the whole field of occurrences of water on the earth was not yet born. We might say that hydrology was in an embryonic stage up to 1930.

The International Association of Scientific Hydrology, organized in 1924, recognized the relationship of surface streams, lakes, snow and ice, and underground waters, but still restricted the term "hydrology" to the study of underground water.

It is almost 21 years since the American Geophysical Union established its Section of Hydrology on November 15, 1930. I consider that act as the first formal recognition of hydrology as a distinct science covering all the waters of the earth.

The importance of this field was emphasized when the American Society of Civil Engineers authorized the establishment of its Hydraulics Division on April 19, 1938 and stated its purpose as "the advancement and dissemination of knowledge relating to the occurrence of water in nature and its behavior in structures, water courses, and underground." You will readily see that three of these four purposes are directly related to hydrology.

FUNDAMENTAL HYDROLOGY

Hydrology, being a new science, provides a large field for many new discoveries by the scientific explorer. Much of this exploration work, as Mr. Fry has pointed out, can be done best in the hydraulic laboratory. Many of the fundamental theories and laws regarding the operations in the hydrologic cycle have not yet been developed. Such basic relationships as evaporation and temperature of natural bodies of water are not fully understood. Much has yet to be learned of the laws affecting soil moisture and the infiltration capacities of soils.

The unit hydrograph method of computing runoff distribution for a drainage basin is a tool, developed empirically, which has come into wide use in recent years. It has no basis in the fundamental laws of science, yet, I believe that research in the hydraulic laboratory can reveal the basic laws by which it operates. Once the laws are fully understood, the method can be more fully developed and the limitations on its use determined.

The hydraulics of stream flow, although it has been studied for many years, is still little understood, especially for streams subject to variable slope conditions. In establishing gaging stations to measure the slope of streams for discharge determinations, it has generally been a practice to install the gages at the most accessible points along the stream. Frequently, little consideration has been given to bends, variable cross-sections and other non-uniform conditions in the slope reach. It is generally assumed that these conditions will be compensated for in the development of the rating for the stream. The difficulty often experienced in obtaining satisfactory slope ratings indicates that the effects of such conditions are not always neutralized in the rating. In this field the hydraulic laboratory has a challenge to determine the effects of various conditions on slope ratings and to establish guides to be followed in selecting the best possible locations for gages with respect to the physical features of the river channel.

EQUIPMENT AND INSTRUMENTATION

One of the more important parts of Mr. Fry's paper dealt with the development of special devices and equipment. The development of new equipment and instruments must keep pace with the advances in hydrologic knowledge. Probably many of the fundamental facts yet to be learned will depend on the use of new or better instruments for their discovery.

Much laboratory work has been expended on the study of evaporation in an attempt to define its relationship to temperature, wind velocity, relative humidity and other factors. Although some relationships have been established, our knowledge is not complete enough to explain some inconsistencies in nature.

The evaporation from "Class A" evaporation pans in this section of the country is much higher in April in relation to temperature than it is during other months. The rate of evaporation also declines much more rapidly than temperature in October. Any analysis of this situation is greatly handicapped by the absence of evaporation data during the months of November to March. Discontinuance of observations at evaporation stations during this period is largely because of the difficulty of making observations during freezing weather. The development of instruments suitable for making year round observations would be a worth while undertaking for the hydraulic laboratory. The development of satisfactory recording instruments for obtaining simultaneous records of water and air temperatures and evaporation losses would also be of value.

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I understand that the Tennessee Valley Authority has had some experience with weighing gages for evaporation observations, but I have not had an opportunity to find out how well they have worked.

For over 50 years the Price type current meter has been the standard instrument for use in stream gaging work. During this period no really basic changes have been made in the meter, and most developments have been confined to the use of better materials in construction. The long use of this type of meter attests its excellence. However, in this electronic age, I believe that a new meter can be developed which will indicate water velocities directly, instead of requiring the operator to count and time the revolutions of the meter bucket wheel and then compare these observations with a meter rating table to obtain velocities. Recording instruments can also be developed to produce graphs of velocity observations. Such instruments can speed up the operation of gaging a stream and provide much more information about the behavior of water velocities.

The installation of recording gages for obtaining continuous records of river stages is very costly because of the large structure required to house the recording instrument, stilling well, flushing equipment for keeping intakes open, auxiliary gages, and to provide sufficient room for servicing and maintaining this equipment. If some inexpensive, easily installed and reliable instrument, not requiring a stilling well, could be developed, the savings in installation costs would provide for many more recorder installations. Such gages could also be moved to new locations without a large loss of investment in a costly structure.

The field for the development of instruments and other equipment for use by hydrologists is wide open and offers an excellent opportunity for laboratory research.

HYDROLOGY AND STATE HYDRAULIC LABORATORIES

Nearly every state university that teaches hydraulics has a hydraulic laboratory. The size and completeness of the laboratory depends upon the emphasis placed on hydraulics at the particular university. Because the engineering profession is broad and covers many specialties, not every university can be expected to specialize in and emphasize the same phase of engineering. To try to cover all phases would only breed mediocrity.

Several universities have developed excellent hydraulic laboratories, but most of them have emphasized the hydraulics of structures. It is good to see that we now have one that is going to specialize in the development of the science of hydrology. I am glad that the State that has taken this step is located so close to Indiana, as I am sure that there will be numerous occasions in the future when I will find it desirable to consult with the laboratory.

C. E. KINDSVATER.*—It might be observed that the only acceptable excuse for my commenting on Mr. Fry's remarks is to present additional examples of cooperation between hydrology and the hydraulics laboratory. Certainly no one is better qualified than he to understand the broader aspects of the subject. For the past 17 years alone, as chief of the TVA's Hydraulic Data Branch, he has administered an organization composed of a major hydraulics laboratory as well as field investigations, river forecasting, and hydrologic investigations units. His organization has contributed much to the successful accomplishment of the Authority's comprehensive river-development program.

Mr. Fry named five areas of cooperation between hydraulics research and hydrology. The first four dealt with instrumentation, having as their primary purpose the improvement of techniques for direct measurement of hydrologic quantities. The fifth area of cooperation was described as "working out solutions for hydrologic problems, either in the realm of hydrologic research or problems encountered in a particular hydrologic project." We may conclude that the last topic includes activities which aim toward the development of new theoretical concepts.

Engineering hydrology is inescapably empirical—to a degree. Direct measurements of basic hydrologic data are the fabric of the science. Every formula is judged by its ability to substantiate an observed occurrence. In other words, good instrumentation and refined measuring techniques will always be of primary concern to the hydrologist. But, experience and measurement is not enough. Real progress depends on the development of new ideas, new physical concepts, and new analytical tools with which to interpret the observed data.

My own preoccupation with the last item in Mr. Fry's enumeration would lead me to place it at the top of the list. In my opinion, adequate solutions of the hydrologist's problems must inevitably begin with a sound physical analysis of the phenomenon concerned. We would probably all agree on this elementary thesis. But, are we doing all we can to demonstrate our faith in this idea?

Let us examine a specific problem which concerns a majority of this group—the movement of water in rivers. The basic problem is that of varied flow in non-uniform open channels. We recognize that existing computation methods are inadequate for all but the simpler aspects of this problem. Equations which describe the mean flow pattern in

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prismatic channels have not been successfully extended to embrace the infinite variety of boundary conditions which characterize natural channels. There is little reason, at the present time, to think that this problem is amenable to exact analytical solution. There is both reason and example to indicate that it is a problem which is deserving of a great deal of basic research.

Research is not necessarily conducted in the laboratory, but this is usually the most economical and expeditious procedure. Field research ordinarily requires large investments in manpower and time. Instrumentation is difficult, and natural obstacles to detailed measurements are often insurmountable. A satisfactory range of test conditions is frequently obtainable only by spreading the research activity over a number of widely separated sites. More often, it is necessary that the research be delayed, perhaps for years, in order that extreme natural occurrences might be investigated. In the laboratory, on the other hand, a full range of flows is obtained by adjusting a valve. One man, working comfortably and with scientific precision, will gather in minutes or hours data which would require several men days or weeks to collect in the field. I need not elaborate, for this, too, is an accepted thesis.

Why is it, then, that more fundamental research in river hydraulics is not underway at the present time? I would like to suggest one explanation. Relatively few of the agencies represented by this group have adequate laboratory facilities of their own. Few states, for example, are as fortunate as the State of Illinois in having excellent facilities such as these we are here to dedicate. Many agencies concerned with different aspects of engineering hydrology will never have, and, in fact, could never justify hydraulics laboratories of their own. These agencies, then, must take their problems to such laboratories as are available and capable of handling sponsored research. And there's the rub! If you are a small, private organization, it may be necessary only that you have in your staff persons capable of recognizing the need for "outside" help. The process of acquiring such assistance may be relatively simple. If, however, you represent a public agency, as most of you do, you know that this process is not infrequently the ultimate stumbling block. To acquire the assistance of others is to admit that your own staff or your own facilities are inadequate. That, in itself, is more than a selfish administrator can permit. Furthermore, sponsored research must be paid for, and this disturbs your budget. Knowing that the top brass might have to be sold on the economic feasibility of research, you might stop here. Or, perhaps, you have a nasty contracting officer, a clerk, or a frustrated lawyer who makes contractual matters so unpleasant that you prefer to avoid the entire issue. Let us hope that it is not your engineering staff who constitute the barrier. It is my opinion that the situation we face calls not only for faith in research but for bold perseverance on the part of our administrators.

I would like, now, to describe a more optimistic trend of events. Mr. Fry mentioned briefly two "indirect" methods of estimating flood flows which were deserving of research. These were the so-called slope-area and contracted-opening methods. It is agreed, at present, that the most satisfactory method of determining discharge in large, natural channels is by direct measurement. Yet, it is recognized that major floods on large rivers are frequently impossible to measure accurately by existing methods, and that flood flows on small streams often subside before an observer can reach the site. In many instances, too, past floods of great significance are recorded, if at all, only in terms of high-water marks. It is on such occasions that the indirect methods of computing flood flows are called upon to salvage the available records.

Agencies primarily responsible for collecting records of flood discharges have become increasingly aware of the importance of indirect methods. They are also aware of the inadequacies of existing computation procedures. For several years the Water Resources Branch of the U. S. Geological Survey has been compiling and analyzing field data which would contribute to a better understanding of indirect methods. They have accumulated considerable evidence to support existing methods of computing flood flows by both the slope-area and the contracted-opening techniques. And yet, among their data, there are those unexplained inconsistencies which continue to throw a shadow of uncertainty over all methods of determination by computation instead of measurement. During the past year the U. S. G. S. has authorized a program of fundamental research on the contracted-opening method. They have taken this problem to the laboratory. Through arrangements with the Georgia Tech Research Institute, they are sponsoring a program of laboratory research which I have been privileged to direct. Here, then, is another example of a way in which the laboratory can be called upon to aid the hydrologist.

You might be interested in knowing how we have attacked this assignment. We have acknowledged the fundamental nature of the problem by describing the project as an investigation of the flow of water through open-channel constrictions. Starting with simple rectangular constrictions in a level, rectangular flume, we attempted first to gain a better understanding of the flow pattern and to isolate the critical variables. An analysis based on these exploratory tests was then substantiated for a larger variety of simple constrictions, and the relative influence of each of the major variables was systematically determined by experiment. Continued success in correlating the effects of the various

boundary conditions was encouragement for extending the basic analysis to the general problem. Until now, we have extended the laboratory investigations to cover three degrees of boundary roughness, several irregular channel shapes, and a wide variety of constrictions, including some which were eccentrically located in the flume, others which were placed at an angle to the flow, some which had sloping sides, simulating embankments, and many others which contributed to a full range of practical shapes. We are optimistic about our accomplishments to date. We are confident that the results of this study will be practicable.

The contracted-opening method, as suggested by Prof. S. M. Woodward, was first described in American literature in 1918 by Mr. Ivan E. Houk. The urgent need for this method of determining flood flows is attested by the fact that it has been kicked around for the past thirty-three years. We can only hope that the manhandling which it has received in the past eight months will leave it in better condition than that in which we found it.

In conclusion, I would like to take this opportunity to appeal for cooperation of another variety. Mr. Fry has given proper emphasis to the need for competent personnel in this field. Fundamental research, in engineering as in pure science, calls for a staff of experts. Experts in hydraulic engineering, for example, are certainly not produced in four years of civil engineering training. And it is increasingly apparent that job experience alone cannot produce a staff of the highest calibre. As our own technology advances, it becomes more complex. Not only in our research laboratories, but in our design and field offices we need more men who have extended their college training to include at least one full year of graduate study. The colleges are attempting to meet this need by expanding their graduate programs. They need your help. Maintenance of competent teaching staffs and adequate educational facilities can hardly be justified on the basis of the small number of students who participate in these programs at the present time. Therefore, one of the greatest professional contributions which each of you can make is to publicize your need for competent hydraulic engineers.

BED-LOAD FUNCTION FOR SEDIMENT TRANSPORTATION IN OPEN CHANNEL FLOWS

BY H. A. EINSTEIN*

WITH DISCUSSIONS BY VITO A. VANONI AND RALPH W. POWELL

Most engineering design is based, to a large extent, on experience. When the backlog of experience in a given field increases in volume, it becomes difficult to apply it to a specific case unless all the individual pieces of experience are organized into a system permitting classification and comparison of the individual observations. The philosophy of such a system is called a Theory, and the means of expression should preferably be the mathematical formula. Today, I shall attempt to give you some of the basic ideas behind my approach to the description of sediment transport.

The starting point of this development is an abundance of flume studies and an extreme lack of river measurements. The flume experiments must be interpreted as a model study of the natural river and are of significance for the river problem only if the laws of similitude can be determined according to which the flume can be tied to the prototype. The basic question of similarity as applied to sediment transport has been in my center of interest for many years. Our basic way of approach is that of reducing all variables and constants to dimensionless form. In every mathematically correct system of similarity will such dimensionless parameters and constants be equal in model and prototype. So will the equations between them, as long as the physical problem is the same. This is the explanation for the use of some rather abstract dimensionless parameters.

The expression "Bed-Load Function" is a direct translation of "Geschiebefunktion" which has been used during the past 20 years in the European literature with the same meaning: the equilibrium rate of bed sediment moving through a river section. In Bulletin No. 1026** a detailed description was given of the exact definition of the problem and of its solution, as we see it today. The treatment of the bed is especially new wherein the constituents of the sediment mixture do not move at equal rates. In order not to just duplicate the Bulletin, I shall give here the basic ideas only, without the use of formulas.

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**Einstein, Hans A., "Bed-Load Function for Sediment Transportation in Open Channel Flows," U. S. Dept. of Agriculture Tech. Bull. No. 1026, Sept. 1950.

My students always ask me why we are interested in the determination of the sediment load in a river, and my answer is that there are mainly two reasons: first, we may want to predict the rate at which sediment must be expected to accumulate in a proposed reservoir; second, a comparison of the rates of sediment, which a river channel is able to transport, with the rates supplied to the channel from the watershed indicates whether or not a given stream channel may be expected to be stable. Both problems thus involve the determination of the sediment capacity of a river channel. The difference is that in the reservoir problem the emphasis is on the bulk of the transport, or in most cases, on the finer constituents of the sediment which are only lightly represented in the bed, while the channel problem is determined by the motion of the larger sized particles which represent the bulk of the bed material. Therefore, it is not sufficient to describe the sediment load as a curve of the sediment rate against discharge or stage, but the size analysis of the transport of various flows must be given, too. In Bulletin 1026 the rates of transport are calculated in function of discharge separately for the individual grain size fractions. These rates may then be added to find the total sediment rates which are required for the solution of a particular practical problem.

If we aim to determine the capacity load of a sediment-carrying channel, the complete flow pattern must be known, as described by the discharge, the water depth, the slope, the average velocity, the velocity distribution and the shear forces along the bed. Much work has been done with good success on the description of river flows, but there are still many unanswered questions left. For instance, different friction factors were determined in the same alluvial stream channels at high and low discharges. Just recently, I published a paper together with Mr. Barbaressa, now in Minneapolis, on just this problem. By calculating friction factors from a large number of measurements in different streams, we found that the friction in those streams can be described as the sum of two parts. One is the friction which refers to the grains whether it is gravel, sand or finer. This part can be expressed in a similar way as, for instance, the results of Nikuradse's experiments were expressed. He measured the friction of flows in pipes which were arti-

ficially roughened by glueing sand grains to the otherwise smooth wall. We found then that it was necessary to add to this friction (which may be predicted from formulas as derived from Nikuradse's experiments) an additional effect which described the frictional effect of the irregularity of the bed, and of the sand bars. This in itself is quite reasonable but was not obvious. The only way to decide whether actually this frictional effect could be divided into two parts was to find out if a reasonable description can be given of the bar resistance. This actually appears to be possible if the bar resistance is described as a function of the sediment transport on the bed. With our meagre knowledge of the sediment transport, this approach seems to be of little help. However, we can use a trick. In Bulletin 1026 it is shown that the part of the sediment transport moving near the bed, which Bagnold has called surface creep, can be given as a function of ψ, the ratio of the grain size to the tractive force along the bed. The tractive force or bottom shear may easily be calculated from the flow.

How must we interpret this additional friction? Is it the energy spent for the transport itself? This question must be answered with a definite negative because the value is the highest at low rates of transport and becomes practically zero at high rates. The only interpretation which did not lead to any contradiction is that which links it to the shape resistance of bars or ripple marks. This can be seen by careful study of the frictional effect in available published experiments. For instance, Gilbert's experiments will show it. The bars seem to disappear, too, at the highest rates of transport. If the friction laws of a flow over an alluvial bed are known, one may predict the average flow velocity and the velocity distribution by known methods.

The Bulletin describes rather elaborately the procedure of the hydraulic calculation on the basis of these rather new formulas. I did this for the sake of the field engineer who is usually not too familiar with the research literature. Only he can check by field measurement whether or not such new methods are applicable to actual field conditions different from those from which they were derived.

After the flow is described in such a channel, it is now necessary to derive the resulting sediment transport. The literature, so far, has given two approaches: one is the description of suspended load and the other a description of bed-load transport, mostly by formulas of the tractive-force type. The big difficulty was that there was actually no link between the two and if you asked yourself how you can use the suspended load theory in order to predict the rates of transport, then you always came back to the fact that no prediction of suspended load was possible without some kind of measurement determining some reference concentration. I started my studies beginning with the bed-load because at least for the stability of the bed, the transport rate along the bed must be expected to be the most important part of the load.

Bed-load was usually studied by experiments in flumes either in the laboratory or outside. Then, one tried to find relationships describing the results. After analyzing all available data I felt it was necessary to enlarge the range, in which experiments of that kind were made, by simultaneously increasing the flow velocities and decreasing the grain size. I got into conditions where a large part of the bed sediment moved in suspension and found that experiments of this kind could not be expressed by the same equations as the experiments which have practically no suspension. From that I concluded that there must be a relationship of the bed-load type governing the reference concentration of the suspension because the total rate of transport which includes suspension and bed load was a function of the flow, but different from that for pure bed-load experiments. So I figured that some new thought must be brought in to explain how suspension was governed. That was done in the following way. It was assumed that the suspended load theory applies over practically the entire depth to describe the concentration. Our description of suspension breaks down only in the immediate proximity of the bed. To see this, let us review how suspension is caused by turbulence. The turbulence gives vertical velocities to individual fluid masses within the flow. These masses take the suspended load particles along and thus lift them vertically. The masses which together move at the same velocity and direction become smaller and smaller very near the bottom as is shown by the theory of turbulence. At a distance of a few particle diameters from the bed these individual masses, which move as a unit, become so small that they can not move the particle any more. They only tickle them somewhat.

At this time we have some experiments running with a light weight material as sediment. In these, one can very well distinguish three layers with respect to the sediment: the bottom layer of particles which do not move; the top layer where the particles move about with the velocity of the water and between the two there is a layer of one to three particles thickness, where they move very much slower by distinctly rolling or sliding over the bottom. This motion suggests the following interpretation. Down to about two diameters from the bed one may apply the suspended load theory. Between that elevation and the bed the transport resembles much more that of bed-load. An attempt to apply the same formula to this bed layer as applied to pure bed-load, in experiments without suspension, turned out to be most successful if the concentration in this layer was introduced as the initial concentration for the suspension above. Thus the composite transport in the entire vertical be-

came a function of the bed-load transport and, thus, of the discharge. In the actual experiment one measures the total transport at the downstream end of the flume. The analysis of the results is easy as long as the sediment consists of uniform grain. First, the transport in the bed layer is predicted from the results of experiments without suspension. This rate defines the bottom concentration. By integration of the suspended load over the full depth and by addition of the bed load, the total load is calculated and may be compared with the measurement. This same calculation applies to the river, too.

Then came the next question: what happens when the sediment is not uniform? If you have ever worked with the suspended load theory you know that the parameters of the calculation are different for the different particle sizes and types. It is thus necessary to calculate the suspension of the individual grain sizes separately and the same applies also to the total load. The vertical distributions of the different suspended grain sizes can be calculated independently, and the same independence was found to be applicable, to a large degree, to the bed-load motion of a mixture. It was shown that for larger particles of a mixture this independence is absolute. For the finer components of a mixture and for grain sizes smaller than 0.2 mm. one must introduce a few corrections. The fine components of a mixture for instance hide behind or between the coarser particles of the bed and are not to the degree available for transport that would be expected by assuming full independence. Also when the particles become so small that the total surface takes the appearance of a hydraulically smooth surface one must introduce some corrections. These corrections are very important if the proper results are to be obtained. In Bulletin 1026 a set of such corrections is shown which has been found to describe reasonably well rather large numbers of flume experiments. But it must be kept in mind that these corrections are empirical and still subject to future improvement. We have found lately, for instance, that there is one case where some modifications of these corrections become necessary. This is the case where the bed assumes the tendency to develop layers. Stratification will occur especially under conditions of deposition. It appears that on a stratified deposit there is less interference between the particles than on a uniformly mixed bed.

In summarizing, one may state that that basic concept of sediment motion on an alluvial bed is successful, which (1) divides the total transport into a bed-load part near the bed; (2) a suspended load part above this layer, where the bed-load motion defines a reference concentration near the bed. It is necessary, furthermore, to calculate separately the transport rates of individual grain sizes within a mixture, mainly because of their different behavior in suspension. More work is necessary on the effective exchange coefficients for suspension in natural river sections. The mutual interference of the various grain sizes of a sediment mixture in the bed-load motion as described empirically in Technical Bulletin No. 1026 needs further checking by river measurements. No available river or flume measurements contradict, in any way, the basic approach to the total load determination as set forth in this paper. Further basic work on this interpretation of sediment motion is in progress.

DISCUSSION

VITO A. VANONI.*—In developing the bed-load function (1)** the author expresses the rate of transportation, q_s, of bed material in suspension by the following formulas:

$$q_s = \int_a^d c_y \, \bar{u}_y \, dy \qquad (1)$$

$$c_y = c_a \left(\frac{d - y}{y} \; \frac{a}{d - a} \right)^z \qquad (2)$$

$$z = \frac{v_s}{k u_*} \qquad (3)$$

where q_s is the rate of transportation of sediment with settling velocity, v_s in weight per unit width of stream per unit time, d is the water depth, $\bar{u}_y$ is the velocity at a distance y above the bed, c_y and c_a are the sediment concentrations in weight per unit volume of the fluid-sediment mixture at distances y and a, respectively, above the bed, v_s is the settling velocity of a sediment grain in still water, k is the von Karman universal constant and u_* is the friction velocity at the bed. The concentration c_a at the reference level y = a in the above equations is obtained from a bed-load equation. The total load or rate of transportation of a given size fraction of sediment is the sum of the suspended and bed-load transport rates, and is given by equation 63 of reference 1,

$$i_T \, q_T = i_B \, q_B \, (P \, I_1 + I_2 + 1) \qquad (4)$$

where q_T is the total transport rate of bed material, i_T is the fraction of total load within the size range under discussion and q_B and i_B have similar significance with regard to the bed load.

Measurements in laboratory flumes (2), (3) and in streams (4) have shown that the form of equation (2) for the distribution of sediment in suspension agrees with observations. But these measurements also show discrepancies between values of z given by equation (3) and those that give the best fit of the experimental data. The writer would like to discuss the method of calculating the exponent z and its effect on the calculated rate of sediment transportation.

It will be noted that in the author's calculations an average value of z is obtained by using the value of 0.4 for von Karman's constant k. Observations in the laboratory show that k tends to decrease when the concentration of suspended sediment increases so that value of z calculated from observed values of k are larger than those calculated with a k value of 0.4. Observations in the laboratory have also shown that the values of z calculated with observed k values are larger than the observed values of the exponent. The observed value of the exponent which will be denoted by the symbol z_1 is the slope of a logarithmic graph of measured values of c_y plotted against the function (d - y)/y.

Fig. 8 shows observed values of k plotted against c_m, the average concentration of sediment in the flow. The data were obtained from observations in a flume 2.75 ft. wide carrying uniform sized sands 0.10 and 0.16 mm. in average grain diameter. It will be seen at once that k decreases with increasing concentration and that the effect of the sediment appears to be greater for the shallower flows although the relationship cannot be defined very well because of the large scattering in the data. The effect of suspended sediment in reducing k was noted by the writer in early experiments (2) on sediment transportation.

Fig. 9 shows calculated values of the exponent z (using observed values of k) plotted against z_1, the observed value of the exponent which is obtained

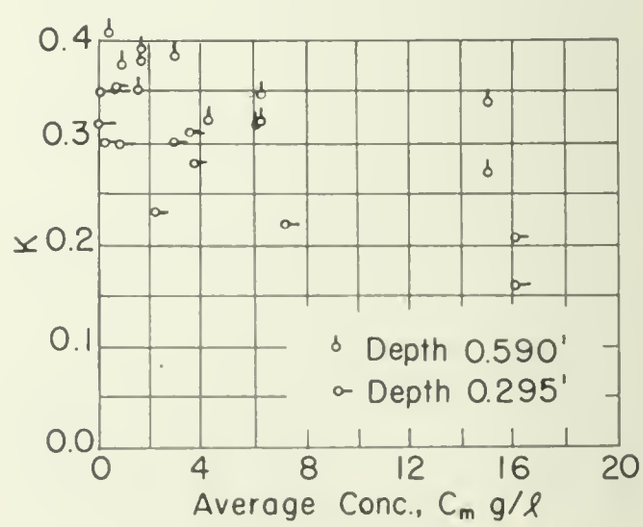

FIG. 8. — GRAPH OF OBSERVED VALUES OF THE VON KARMAN UNIVERSAL CONSTANT k AGAINST THE AVERAGE CONCENTRATION OF SEDIMENT IN WATER FLOWS OF TWO DEPTHS.

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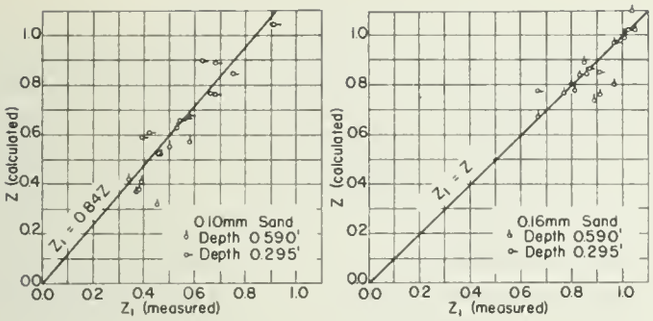
**Reference number in BIBLIOGRAPHY at end of this discussion.



(a) (b)

FIG. 9.—GRAPH OF CALCULATED EXPONENT z FOR DISTRIBUTION OF SUSPENDED SEDIMENT AGAINST OBSERVED EXPONENT z_1.

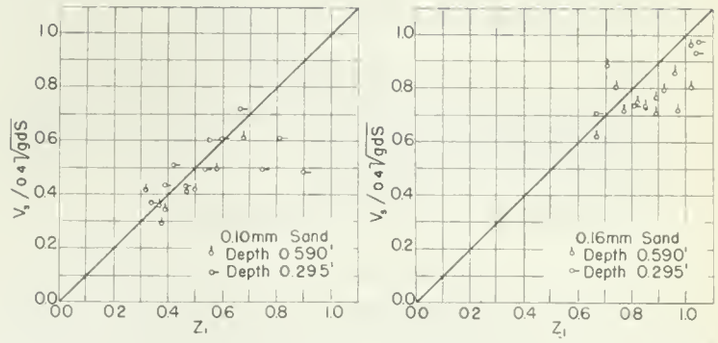


FIG. 10.—GRAPH OF THE QUANTITY $v_s/0.4\sqrt{gdS}$ AGAINST THE OBSERVED EXPONENT z_1 FOR DISTRIBUTION OF SUSPENDED SEDIMENT.

from measurements of the distribution of suspended sediment. Data for 0.10 mm. sand is shown in Fig. 9a. The straight line which roughly fits the points indicates that the ratio of z_1 to z is about 0.84. The graph also indicates that this ratio is higher for the deep flows (0.590 ft.) than for the shallow flows. The results of experiments with 0.16 mm. sand shown in Fig. 9b indicate that the observed and calculated exponents are about equal. Ismail (3) observed variations similar to those described above in the exponents z and z_1 in sediment transportation experiments made in a rectangular pipe 10.5 in. wide and 3 in. deep. He found that the ratio of observed to calculated values of the exponent was 0.67 for 0.10 mm. sand and 0.77 for 0.16 mm. sand. The above data indicate that the ratio of the exponents varies with sediment size and hydraulic parameters although the reason for the variation is not clear.

Fig. 10 shows graphs of z, calculated with a k value of 0.40, plotted against z_1, the observed exponent, for the experiments from which the data of Fig. 9 were obtained. The diagonal lines on the graph indicate the relationship $z = z_1$ and approximately fit the data for both sediments, although the scatter appears to be greater than in Fig. 9 where measured k values were used to calculate z. In view of the lack of knowledge of the variation of z it appears from the above results that the author's method of calculating z using k = 0.40 is a reasonable approximation. On the other hand observations by Anderson (4) on a small stream indicate that the agreement between z and z_1 is not good even when z is calculated with k = 0.40. His measurements show a ratio between z_1 and z of about 0.5 for sediments ranging in size from 0.1 to 0.5 mm.

In view of the possibility of a large error in z it is of interest to see what effect such an error can cause in the calculated sediment transportation rates. This was done using tables 6 and 7 of the author's reference (1) and assuming that an error in z would not affect the rate of movement in the bed layer or the friction velocity. These calculations show that for values of z in the neighborhood of unity an error of 20 per cent will cause errors in calculated rates of transportation of as much as 100 per cent. The error in the transport rate is reduced as z is increased so that at z of approximately 2.5 the error in the transport for a 20 per cent error in z is only about 20 per cent. The error increases as z decreases.

The above results are presented in an attempt to point out the need for additional basic information regarding the characteristics of sediment laden stream flows. Once such information becomes available it can be incorporated into the author's theory to improve the reliability of results obtained with it. The fact that some of the hydraulic quantities such as z are not known precisely need not detract from the author's theory. Actually the theory serves to emphasize the importance of some of the basic quantities thus bringing them to the attention of research workers and accelerating their clarification.

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RALPH W. POWELL.*—Some years ago an unknown author wrote the limerick:

"There's a wonderful family called Stein,
There's Gert and there's Epp and there's Ein;
Gert's poems are bunk,
Epp's statues are junk,
And no one can understand Ein."

I am sure you realize that was not written about our speaker this afternoon—you probably know it referred to his father—because Dr. Einstein has presented this subject very clearly. His pamphlet is not easy reading; it is all there and you can dig it out, but I think you will admit that he has tackled a difficult subject. A colleague of mine remarked that he had gone into a harder field than his father.

Both Dr. Einstein and Dr. Vanoni are experts in this field who have devoted years to its study. In contrast I am only a general practitioner of hydraulics who has spent the last two weeks trying to absorb the highlights of what Dr. Einstein and others have written on the subject. In reading the paper I have discovered several places where it seemed to me there were misprints or minor numerical errors. These are unavoidable in a work of this character and magnitude, but I would suggest that the author take advantage of the printing of the proceedings of this conference to publish a list of corrections.

This paper represents the culmination of 18 years of work and Dr. Einstein is to be congratulated for having developed a method which will give a definite answer to a puzzling engineering problem.

The question as to how accurate the answer is, may be approached in two ways. One is to investigate the accuracy of the various assumptions on which the development was based, and the other is to compare the final computed result with actual measurements such as were made on the Enoree River in South Carolina (A).[1] I wonder if the author has made any such test.

As to the assumptions, I would question several. Eqs. (1),[2] (2), and (3) were originally derived for pipes, and I am not sure they apply to open channels. I realize that El-Samni (author's reference 8) checked Eq. (2) within 2 or 3% (by subtracting 0.3D from his measured y's), but my own experiments on an artificially roughened rectangular channel (B) led to the equation $C = 17.9\beta_k + 41.2 \log_{10} (R/E)$, which gives $\bar{u}/u_* = C/\sqrt{g} = 3.16\beta_k + 7.27 \log_{10} (R/E)$. Here E is a measure of the roughness on a somewhat different scale than K_5. Elsewhere (C) I have shown that experiments made on this campus with a smooth triangular flume can be represented by $C = 19.49 \log_{10} (\underline{R}/C) + 60.1$, which reduces to $\bar{u}/u_* = C/\sqrt{g} = 10.60 + 3.44 \log_{10} (\underline{R}/C)$. These results indicate that the value 5.75 that has been so generally used, is not a universal constant, but depends on the shape of the cross-section, and can vary at least through the range 3.44 to 7.27.

As the author admits on page 41,[2] Fig. 4 is based on uniform sand grains. The changes where the particles are of various sizes but quite uniformly mixed, or where the various sizes are not uniformly mixed, are shown in Figure 11.

The abscissae are proportional to Dr. Einstein's k_s/δ, and the ordinates are proportional to his x. The points are from Nikuradse's data for particles of uniform size, the line labeled "rough" is for a uniform mixture of various sizes, and the "Colebrook transition" is for a non-uniform mixture.

One is naturally curious as to how Figure 5[2] was obtained. On page 41 the author admits that its application is limited. Similar questions arise as to some of the other curves (7 and 8), and as to the statements that the bed layer is two grain diameters thick, and that A_L or $\lambda = 100$.

Near the top of page 31 the statement is made that "the particle moves if the instantaneous hydrodynamic lift force overcomes the particle weight," and at the top of page 36 the probability of movement is taken as the probability of the lift force exceeding the submerged weight. It would seem to me that movement is by rolling or sliding, and that this would almost always occur before the lift equalled the weight. Also it should be pointed out that any slope of the bottom, either longitudinal or transverse, makes it easier to move the particle.

I believe that the suggestion made by Dr. Rouse (D), that "the ratio of volume of sediment transported to volume of flow may be a definite function of only three dimensionless expressions," should be investigated further. The three expressions he suggests are a sort of Reynolds number based on grain size and friction velocity, the ratio of the fall velocity to the friction velocity, and the standard deviation of the fall velocity. I would also urge the study of a paper by Paul Nemenyi (E), which displays an amount of learning on this subject rivaling that of our author.

Finally, it is perhaps unnecessary to point out that even if this paper is a satisfactory solution of the problem of bed-load movement as a steady state, there still remains the equally important matter of unsteady flow, the rate of erosion or deposition when either is occurring.

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[1]Reference letter in BIBLIOGRAPHY at end of this discussion.

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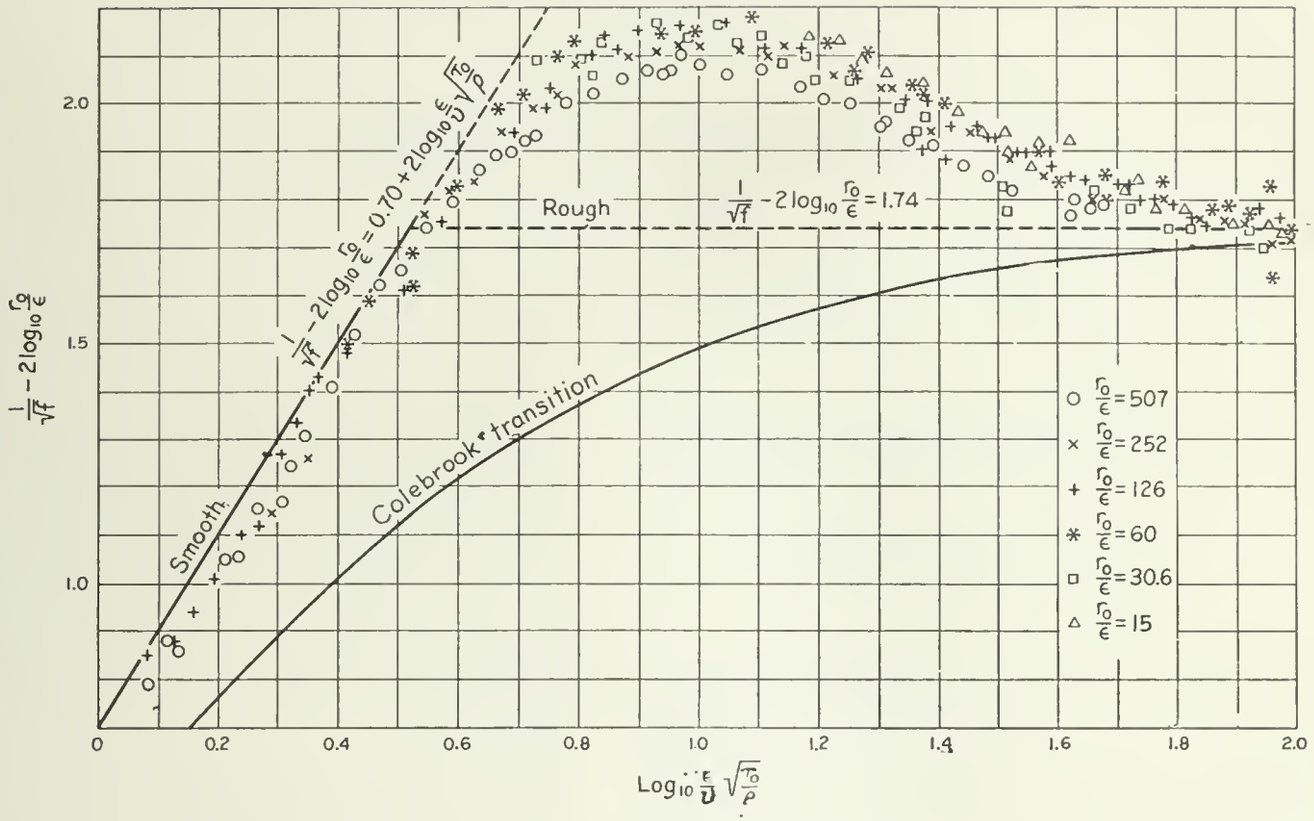

FIG. 11.

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SEDIMENTATION PROBLEMS

INTRODUCTION

BY L. C. GOTTSCHALK*

Sedimentation is one of the critical factors in the development, use, and management of water resources. This has long been recognized in Illinois. No state agency has carried out a more constructive program of sedimentation investigations nor over a longer period of time than has the Illinois State Water Survey Division. All of us who have worked in this field have come to recognize the pioneering and valuable contributions made by this agency. Their initial survey on Lake Decatur in September 1931—more than 20 years ago—is the first such survey ever made by a state agency. Their continuing interest in sedimentation is quite evident from the number of papers on sedimentation included on the program of the Conference on Water Resources in connection with the dedication of the Water Resources Building.

During the past 5 years I have worked very closely with the staff of the State Water Survey Division in connection with a sedimentation program the Water Survey Division is carrying on in Illinois in cooperation with the Soil Conservation Service and the Illinois Agricultural Experiment Station. Under this program detailed sedimentation surveys have been made on 17 reservoirs. Four reservoirs surveyed in the 1930's have been resurveyed and several additional reconnaissance-type surveys have been made. These surveys cover reservoirs from farm pond size to the largest reservoirs in the State. The watershed areas range from 55 acres to 906 square miles. Rates of sediment production vary from less than 250 tons per acre to more than 5000 tons per acre per year. Annual rates of storage loss range from 0.3 per cent to 2.3 per cent. More than half of the reservoirs surveyed are losing in excess of 1 per cent of their capacity annually. Only five have rates of less than 0.5 per cent. Many of us have considered that in areas where reservoir sites are definitely limited, such as in Illinois, 0.25 per cent annually represents an intolerable rate of depletion of essential, irreplaceable natural resources.

It is of interest to consider for a moment some of the factors in Illinois that are conducive to such alarming rates of storage depletion in reservoirs. Three are of major importance. They are:

1. Character of soils.
2. Intensity of land use.
3. Occurrence of rainfall.

More than 80 per cent of the State is blanketed with loess of variable thickness. Comparison of sediment production rates over the United States from all kinds of soils and soil materials shows that loess has the highest erodibility over wide areas.

The intensity of land use for row crops in Illinois is among the highest anywhere in the country. About one out of every three acres in the State is annually producing corn or soybeans. About two out of every three acres are cultivated. Census figures show that for every acre of hay in the State there are two acres of small grains and four acres of row crops. The land capability classification in the State shows that about 60 per cent of the land is suitable for cultivation. At the present time 68 per cent is being cultivated.

Maximum rainfall intensities are concentrated in a period when the soils are most exposed to erosion in the early part of the cropping season. Here at Urbana, for example, during the 9-year period 1941-1949, 77 per cent of the annual soil loss occurred during the months of May and June.

Extension of the useful life of existing reservoirs and preservation of the remaining reservoir sites will depend in large measure on the speed and success of conservation work on the land. Many reservoirs are already so far depleted and are losing their capacity so rapidly that there is no hope of saving them. Experimental studies and evidences from repeated watershed surveys in various parts of the country have clearly shown that rates of sediment production can be reduced 75 or 80 per cent by application of moderate conservation farming methods based on use of the land within its capability. This will be accomplished, however, only when the local people, both those working the land and those in urban areas, become fully conscious of what can be done and what must be done to maintain their natural resources.

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COLLECTION OF BASIC DATA ON SEDIMENTATION

BY GUNNAR M. BRUNE*

This paper will be divided into four main parts: (1) Types of damage caused by sediment, (2) Sources of this sediment, (3) Methods used in field surveys of sediment damage and sources, and (4) Types of control measures for reducing sediment damage.

TYPES OF SEDIMENT DAMAGE

Sediment causes many different types of damage. All of you here are undoubtedly familiar with the damage caused to water supply reservoirs by sediment. Many cities in this area, like Decatur and Pittsfield, Illinois, are faced with an ever-increasing demand for water, and at the same time a constantly decreasing reservoir capacity because of silt deposits. Sooner or later a drouth strikes, and the city runs out of water. Recreational reservoirs are faced with the same problem.

Power reservoirs likewise suffer. Normally, power reservoirs may store water during wet seasons and use it during dry seasons by drawing down their operating head. If the reservoir becomes filled with sediment, however, water cannot be stored and power can be generated only on a run-of-the-river basis.

Farm ponds may also be destroyed in just a few years by sedimentation. Sedimentation of reservoirs and ponds causes an estimated annual damage of $46,000,000 in the United States, as computed from studies made in various parts of the country.

A second form of sediment damage is infertile overwash on bottomland soils. Figure 12 shows an example of this type of damage along the Skunk River in Iowa, where sand has destroyed or seriously damaged productive agricultural land.

Sediment may not only damage or destroy land permanently. It may also kill crops which grow on the land. Many plants, like tobacco, can withstand considerable flooding by clear water, but are killed if a thin film of sediment is left on the leaves by the flood waters.

When stream channels become clogged with sediment, floods become more and more frequent and rise to higher levels. This problem plagues Galena, Illinois, and hundreds of other cities in this area.

Sometimes, because of channel filling, land may not only become more frequently flooded, but may become permanently swamped. On Hay Creek in Minnesota, swamping has caused complete abandonment of the productive bottomland, which has been allowed to revert to willows and brush.

Drainage ditch sedimentation, as you know, is one of the commonest and most costly forms of channel filling in the Midwest. The estimated annual cost of removing sediment from these ditches is $18,000,000.

Closely related is the filling of navigation channels in our rivers and harbors with sediment. In the 1850's, Galena, Illinois, was a bustling river port, with as many as 12 or 15 steamboats in port at once. The channel was more than 300 feet wide and 15 feet deep. After 1850, dredging became increasingly necessary to maintain navigation. By 1910, half of the channel had silted up completely. In 1916, dredging operations were abandoned. By 1940, the channel was hardly large enough to pass a rowboat. Navigation of the river has ceased, and flood damage is now a serious problem in Galena, because of the increasingly limited capacity of the channel to carry off flood waters.

Sediment damage to highways can be seen along almost any road in the Midwest. Continual cleanouts of roadside ditches are a constant expense to State and County. Clogging of culverts and channels underneath bridges, as shown in Figure 13, requires constant cleanouts or raising of the entire roadbed. Railroads are faced with the same problems.

Sediment often causes great damage to farm property. Fences are buried and must be rebuilt frequently.

FIG. 12.—INFERTILE OVERWASH ON SKUNK RIVER BOTTOMLAND, IOWA.

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FIG. 13. —CHANNEL BENEATH BRIDGE, CLOGGED WITH SEDIMENT, IOWA.

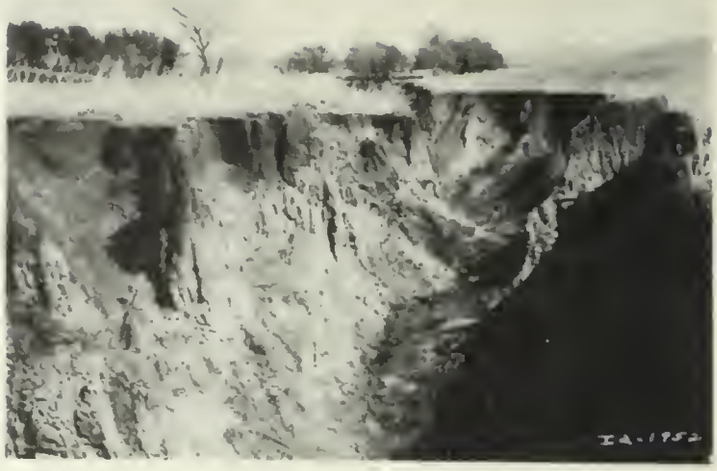

FIG. 14. — A TYPICAL VALLEY TRENCH IN THE LITTLE SIOUX WATERSHED OF WESTERN IOWA.

Cities also are not exempt from sediment damage. Council Bluffs, Iowa, must clean up mud from streets, sidewalks, and homes after every flood. If the sources of this sediment are allowed to go unchecked, the results can be disastrous. The ancient city of Cuicul, near Rome, Italy, once great and prosperous, was later found by Dr. W. C. Lowdermilk of the Soil Conservation Service to be covered completely, except for three feet of a single column, by erosional debris washed off wheat fields on the surrounding hills. It has now been partially excavated by archeologists.

Fish may be strangled to death by silt which fills their gills. But sediment also kills off the desirable game fish more slowly and insidiously in other ways. Mud destroys fish spawning grounds. Silt-laden water kills the vegetation upon which fish depend indirectly for food. Sand fills up the deep pools which once served as a refuge for trout during dry spells.

There are other damages associated with muddy water. You are all familiar with the greatly increased costs of filtration in order to remove sediment from our drinking water. You may not also have realized that the sediment suspended in flood waters sometimes adds enough bulk to the discharge to make the flood crest significantly higher.

All of these sediment damages cost the people of this country an estimated $250,000,000 every year, as computed from studies in various parts of the country. We feel these damages in different ways. Our taxes are higher, in order to pay for dredging silt from navigable channels and streams and cleaning out roadside ditches. Our water bills are higher, in order to cover added filtration costs and the cost of replacing reservoirs filled with silt. We find our food costs going up, partly because sheet and gully erosion, infertile overwash, swamping, and other sediment damages have reduced crop yields. Our electricity costs more, to pay for replacing silted-up power reservoirs. There is less good fishing than there used to be. These are only some of the ways in which sediment damage makes all of us pay, every year.

SOURCES OF SEDIMENT

Where does all this sediment come from? Let's look at some of the sources. In the more humid eastern part of the country, sheet erosion on cultivated land is usually the most important single source of sediment. Wind erosion is another source

FIG. 15. —STREAMBANK EROSION ON THE CUYAHOGA RIVER, OHIO, CONTRIBUTING TO THE SEDIMENTATION OF CLEVELAND HARBOR.

of sediment, although usually not so important in the Midwest.

Gullies often contribute tremendous quantities of sand or other sterile material to the downstream sediment problem. In addition to being an important source of sediment, gullies cause the abandonment of productive fields for crop production.

Valley trenches like the one shown in Figure 14 are similar to gullies except that they are formed in alluvial rather than upland soils. They consist of the deepening of a stream channel or its extension headward into bottomlands or colluvial areas which previously had no channel.

Streambank erosion is another important sediment source in many cases (Figure 15).

Floodplain scour is another bottomland source of sediment.

Road cuts and ditches are a significant source of sediment in some watersheds. Mine wastes are sometimes important.

Other sources, such as landslides, industrial wastes, gravel washing operations, and eroison associated with construction activities, may be locally significant.

Usually there is a change in sediment sources going downstream, with the bottomland sources such as streambank erosion and valley trenching becoming more important and the upland sources such as sheet and gully erosion decreasing in importance. Figure 16 shows the change in the Sangamon River watershed of Illinois. Consequently the control of sedimentation in a large reservoir draining several thousand square miles may pose an entirely different problem than the control of silting in a farm pond in the same area.

Rates of sediment production in the Midwest vary considerably. They are affected by many factors, such as size of drainage area, climatic characteristics, topography, erodibility of soils, and density of vegetal cover on the land. Figure 17 shows the extent of general classes of sediment production in the Midwest. Incidentally, this area includes the extremes in rates of sediment production for the entire country, from the Missouri Basin Loess Hills to the Laurentian Upland.

Figure 18 shows the range in rates of sediment production in each of the 16 physiographic areas shown in Figure 17. Note that although the rates of sediment production vary from one area to another, there is a very great range within each area, caused by the factors mentioned previously.

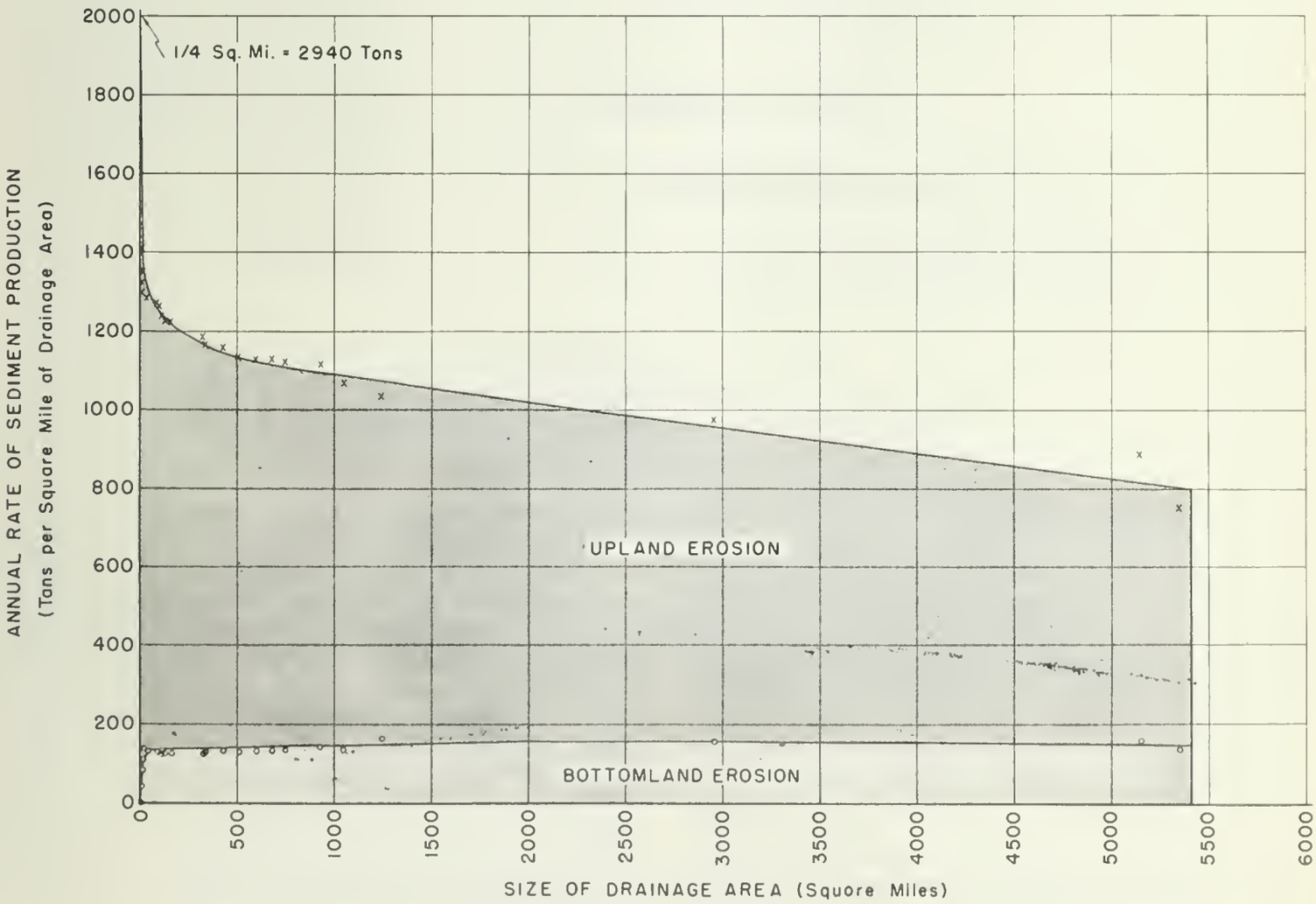

FIG. 16. —SEDIMENT SOURCES AS RELATED TO SIZE OF WATERSHED, SANGAMON RIVER, ILLINOIS.

SEDIMENT RECORDS

- ● RESERVOIR SEDIMENTATION SURVEYS
- ▲ SUSPENDED SEDIMENT MEASUREMENTS
- ■ COMBINED RESERVOIR AND SUSPENDED SEDIMENT MEASUREMENTS

PHYSIOGRAPHIC AREAS

1 GLACIAL LAKE BEDS (Silts and Clays)
2 GLACIAL LAKE BEDS (Sands)
3 LAURENTIAN UPLAND
4 CALCAREOUS WISCONSIN TILL PLAINS
5 WISCONSIN TILL SILTPAN SOILS
6 NON-CALCAREOUS WISCONSIN TILL PLAINS
7 ILLINOIAN TILL CLAYPAN AREA
8 KANSAN TILL WITH SOME LOESS CAPPINGS
9 UPPER MISSISSIPPI LOESS HILLS
10 MISSOURI BASIN LOESS HILLS
11 OSAGE PLAINS
12 OZARKS
13 SANDSTONE AND SHALE HILLS
14 HIGHLAND RIM
15 BLUEGRASS AREA
16 COASTAL PLAIN LOESS HILLS

FIG. 17. —PHYSIOGRAPHIC AREAS OF THE MIDWEST, SHOWING LOCATION OF SEDIMENT RECORDS.

METHODS OF MAKING FIELD SURVEYS

Studies of sediment problems in the field involve a great deal of special equipment and techniques. One of the most common types of field study is the reservoir sedimentation survey. The easiest time to make such surveys is when the reservoir is full of water.

Topographic surveys of reservoirs may be made where an earlier contour map of sufficient accuracy is available. Usually, however, range-type surveys are used. After a map of the lake has been prepared, a series of ranges is laid out, roughly perpendicular to the axis of the lake or tributary arm. The range ends are later marked with concrete monuments for permanent reference, as shown in Figure 19.

Where no previous surveys have been made, it is necessary to measure both the present depth of water and the thickness of sediment deposited since the reservoir was formed. To do this, a rod called a spud is used. This rod, Figure 20, is thrown straight down into the water. It easily penetrates the soft sediment on the bottom and partially penetrates the "old soil" beneath. The cups on the spud bring up a sample which shows the sediment thickness. A sounding line attached to the spud is used to measure present water depth.

During the survey the range ends are marked with flagging. Usually a man is stationed at one

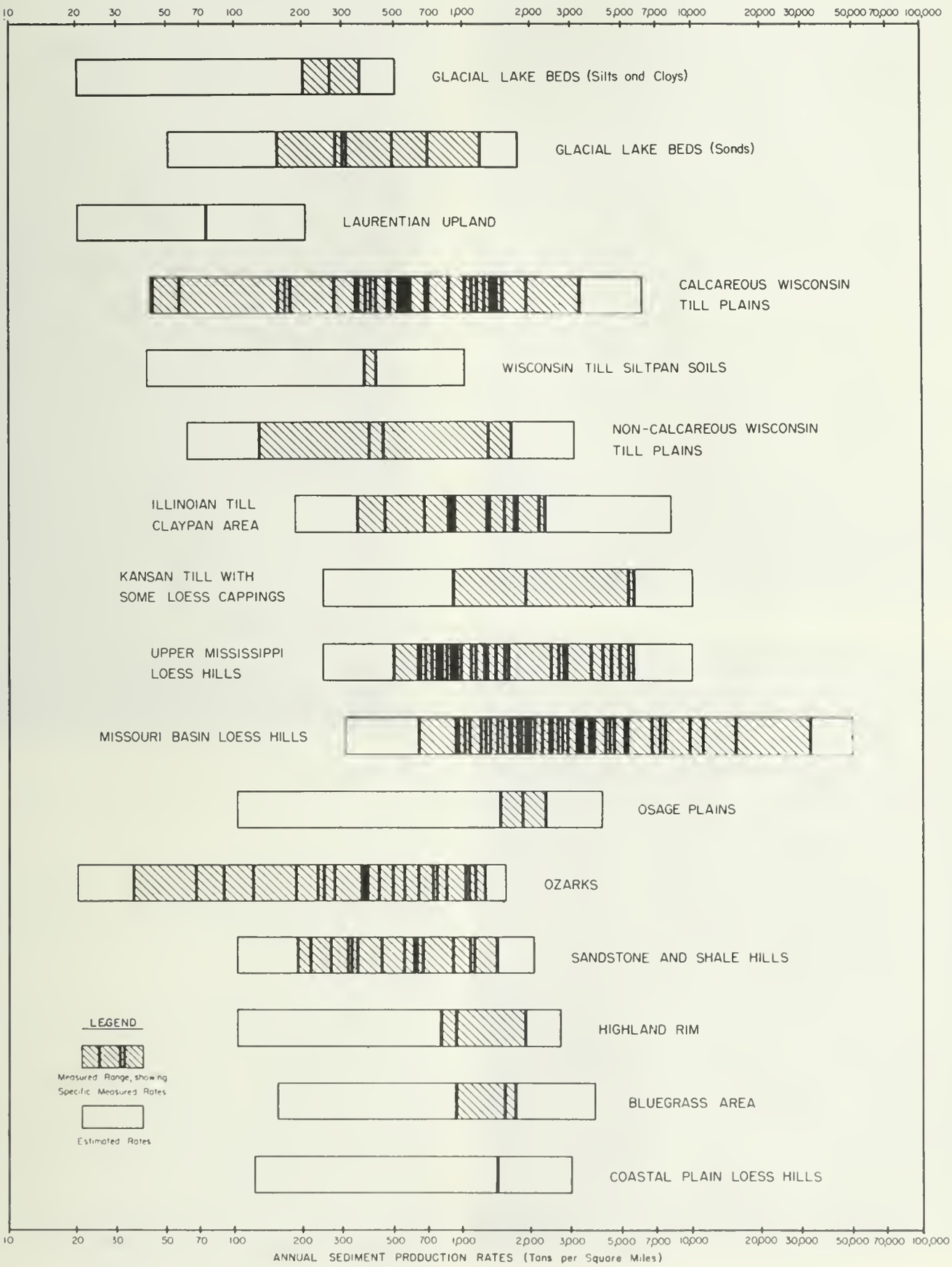

FIG. 18. —RATES OF SEDIMENT PRODUCTION IN MIDWESTERN UNITED STATES.

range end with a transit, and keeps the boat on line by hand signals as it crosses the reservoir.

At another point on the shore, from which the entire range can be seen, another man is stationed with a planetable, Figure 21. He has marked the approximate points on his map where measurements should be taken, and when the boat, proceeding along the range, approaches one of these points, he raises a flag. When the boat reaches the point, the flag is dropped, and the spud is dropped into the water. The line is pulled taut straight up, the water depth is read, and the planetable operator marks the exact location of the shot on his map. Then the spud is pulled up and the sediment depth and penetration into the bottom are recorded.

On resurveys, it is unnecessary to measure the sediment thickness with a spud, and soundings are taken along the same ranges with a sounding weight. For resurveys on larger reservoirs, echo-sounding equipment greatly expedites the work.

In some cases reservoirs are partially drained for the survey. In such cases, the ranges or cross-sections may be run on dry land, using stadia for distances. A soil auger, Figure 22, with extensions up to 30 or 40 feet is used to determine the sediment depths. Trouble is sometimes encountered in pulling such long augers out of the sediment, and some sort of jacking device may be necessary.

FIG. 19.—RANGE END MONUMENT, TEMPORARILY MARKED WITH FLAGGING.

FIG. 20.—SIX-FOOT SEDIMENT SPUD, SOUNDING LINE, AND SOUNDING WEIGHT.

In other cases, reservoir surveys may be made through the ice in winter. Such surveys can be made more accurately than on the water, but many cold-weather difficulties are encountered, such as freezing of sediment on the spud so that it cannot be cleaned off.

Figure 23, taken during the sedimentation survey of Lake Rockwell, Ohio, shows the use of a 20-foot sounding pole. Such a sounding pole may be used for winter surveys through the ice, or for summer surveys on shallow lakes. An ice chisel must be used to cut holes in the ice for measurements during winter surveys. With the sounding pole it is necessary to "feel" for the top of the very soft sediment on the lake bottom. Then the rod is pushed through the sediment to the hard "old soil" or channel beneath. The difference between the two readings is the sediment thickness.

In reservoir surveys, sediment samples are required for mechanical analysis and volume-weight determination. A number of volume-weight samplers

FIG. 21.—PLANETABLE OPERATOR SIGHTING IN POINT OF MEASUREMENT.

FIG. 22.—EXTENSION AUGER USED FOR SURVEYS OF DRAINED RESERVOIRS.

have been devised, none of them entirely satisfactory. The object is to bring up a core of relatively undisturbed sediment of known volume, which may later be dried and weighed to determine its specific weight. The core of sediment is removed from the sampler, placed in another container, and shipped to the laboratory for mechanical and volume-weight analysis.

Information on rates of sediment production can be gathered by suspended load sampling as well as by reservoir surveys. In Figure 24 is shown the DH-48 hand sampler.

This sampler, which is commonly used on small streams, is of the depth-integrating type. A certain amount of time is required for the sampler to fill at different velocities during which time it is lowered from the surface of the stream to the bottom, and back to the surface, in order to obtain an average sample for all depths. The pint milk bottle containing the sample is then removed from the sampler and labeled.

The main objection to suspended load samplers is that they cannot sample the bed-load which rolls along the bottom of the channel. Bed-load may comprise as much as 80 percent of the total sediment load of a stream. To measure bed-load requires cumbersome sampling equipment or complex streambed installations. The latter has only been attempted on a few streams in the country. Figure 25 shows the installation built by the Soil Conservation Service on the Enoree River of South Carolina. Bed-load traps are located between the vertical vanes in the river bed. The bed sediment was pumped from the river bed to the pump house and settling tanks in the background.

Automatic sediment samplers are often used on small watersheds. One type consists of a pipe with a slot on the top, mounted below a weir notch structure, which intercepts a certain aliquot of the total flow, including water and sediment, suspended and bed-load, and carries it off to a tank where the sediment settles out and may be measured. Automatic samplers are desirable on small streams, where flash floods take place so quickly, often during the night, that there is not time to reach the station and take hand samples.

To measure land damage by sediment, such as infertile overwash, it is necessary to bore into the sediment with an auger. Barrel-type augers are usually preferred because they bring up an undisturbed core of the sediment and "old soil" beneath. The relatively fertile "old soil" which is buried beneath sand, gravel, or sterile subsoil silts, can usually be recognized by its darker color and crumb structure. Buried fence posts and partially buried trees also help in determining the depth of modern

FIG. 23. —USE OF SOUNDING POLE ON LAKE ROCKWELL, OHIO. BOAT IS BEING WINCHED ACROSS LAKE ON "TAG LINE."

deposition, since white men have settled the area. The percentage damage to the land is estimated by comparing crop yields on undamaged areas with those on areas which have been damaged by infertile deposits.

In studies of sediment sources, our soils men determine the severity of sheet erosion in various parts of the watershed in question. By boring into the soil with augers, they can tell how much of the virgin soil profile remains and how much has been washed away in modern times. If the original depth of topsoil of a certain soil type is six inches, and only 3 inches remain, the soils man knows that 3 inches have been eroded off since clearing and cultivation by white man began.

Studies of gully and valley trench growth and streambank erosion are often made by rephotographing areas previously photographed 10 or 12 years ago. Detailed ground surveys may also be made.

Such detailed surveys have been made over a period of years on the Myers valley trench shown in Figure 26, one of the trenches which contributes to the sediment problem in Council Bluffs, Iowa. The rate of trench growth varies from year to year, depending upon the degree of saturation of the soil, intensity and frequency of rainfall, vegetal cover, and other factors. In this case, the rate of growth increased from 0.5 acre per year in the 1938-1944 period to 1.45 acres per year in the June 1947-June 1948 period, and then dropped off again to 0.77 acre per year in the June 1948-May 1951 period. Maximum rate of sediment production from this trench was over 123 acre-feet per square mile of drainage area per year from June 1947 to June 1948.

The Soil Conservation Service in this region has made extensive use of the author's personal plane, a 4-place Stinson Voyager, for rephotographing gullied areas and streambank erosion to determine rates of erosion. They have also used it to obtain photos of reservoirs for sedimentation surveys, in cases where the regular aerial photos do not show the reservoir because they were taken before the reservoir was constructed, and to study the growth of delta deposits in reservoirs.

CONTROL OF SOIL EROSION AND SEDIMENT PRODUCTION

What can we do to reduce high rates of soil erosion and sediment production? In eastern France, the farmers each year dig up the bottom furrows of the field, load the soil into carts, and haul it back to the upper edge of the field. This is a laborious and expensive way of controlling erosion. Still, is it much different from our own practice of allowing sediment to accumulate in reservoirs, navigation channels, drainage ditches, and roadside ditches, and then dredging or digging it out at great expense? Let us consider each of the major sources of sediment separately, to see just how effectively each can be controlled.

In regard to sheet erosion, experiment studies show that contour farming as against straight-row farming will reduce the soil loss by 10 to 60 percent, depending on the slope. Contour listing is

FIG. 24. —DH-48 HAND SEDIMENT SAMPLER.

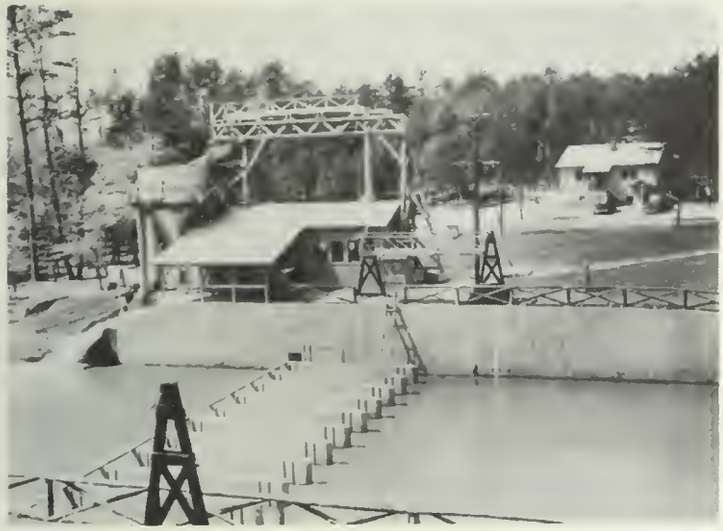

FIG. 25. —BED-LOAD MEASUREMENT INSTALLATION ON ENOREE RIVER, SOUTH CAROLINA.

even more effective. Strip cropping will reduce sheet erosion by 50 to 75 percent, Figure 27. Grass waterways act as very effective filter strips in straining out and catching most of the soil washed from adjacent slopes. Terraces reduce erosion losses by 65 to 95 percent, depending on slope of land and whether the terraces are level or graded. Diversion ditches greatly reduce soil losses on the lower portion of a long slope, by intercepting runoff. And of course, where it can be done without disrupting the agricultural economy, retirement of cropland to pasture or woods will practically eliminate any further sheet erosion.

Wind erosion can be largely prevented by stubble mulching, wind strip cropping, ridging, or windbreaks. In some cases it may be necessary to reseed the area to natural grasses and retire it from crop production.

Gullies can often be stabilized by sloping in and sodding or seeding. In other cases tree planting and fencing to keep out the stock are very effective.

Large gullies or valley trenches may require more expensive control measures, like the stabilizing structure shown in Figure 28. In some cases, a gully-stabilizing structure may also serve as a desilting basin, farm pond, and floodwater detention structure. Any gully can be completely stabilized. The cost is sometimes high, but the benefits are usually higher.

Streambank erosion is a little more difficult to control, but it can be done where the benefits, in the form of prevention of both land destruction and downstream sediment damages, justify erosion control. Types of control include willow matting, timber jetties, riprap, "pipe and wire" revetmets, and "jacks." Side-cutting of drainage ditch banks may be eliminated by sloping the banks properly

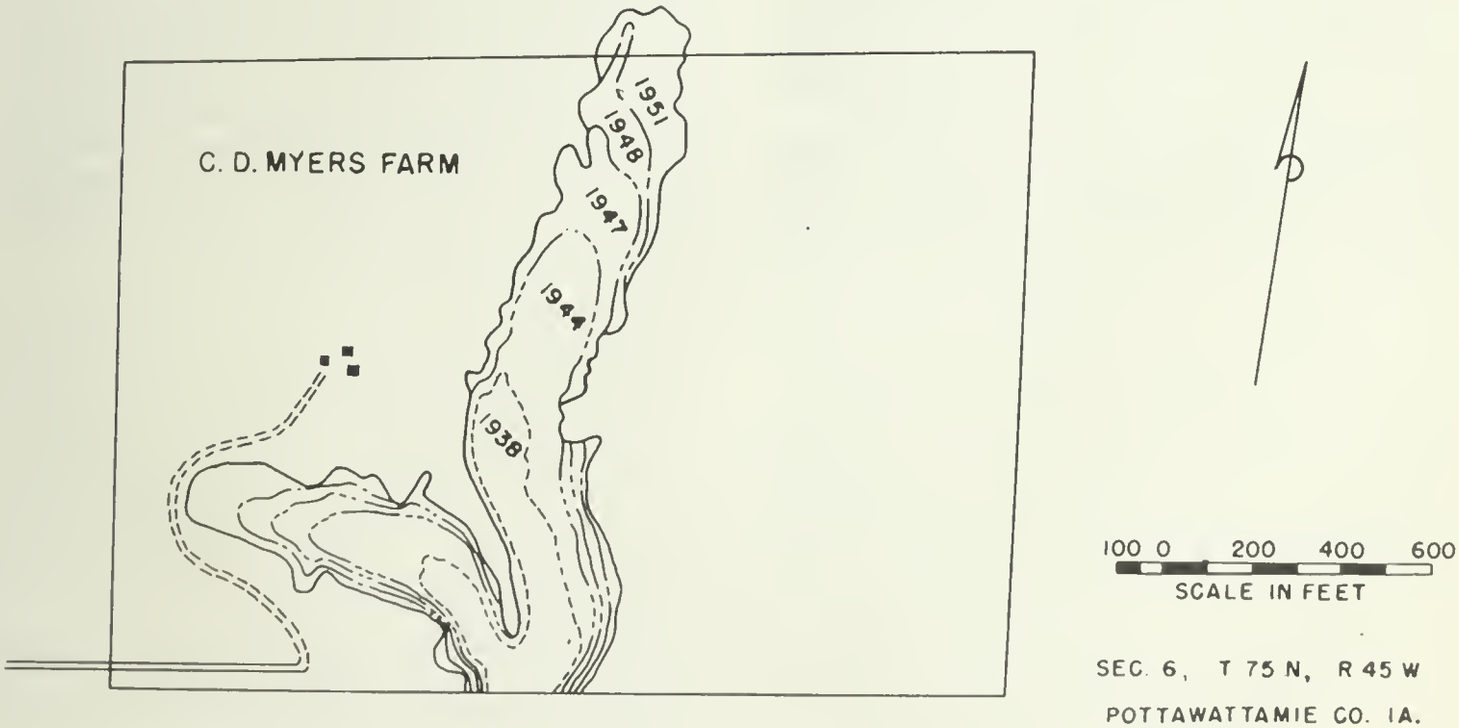

FIG. 26. —GROWTH OF MYERS VALLEY TRENCH, NEAR COUNCIL BLUFFS, IOWA.

FIG. 27. —STRIP CROPPING IN OHIO, AN EFFECTIVE METHOD OF REDUCING SHEET EROSION.

FIG. 28. —NOTCH-TYPE GULLY STABILIZING STRUCTURE, LITTLE SIOUX WATERSHED, IOWA.

and planting to grasses.

To prevent floodplain scour, it is sometimes necessary to construct dikes across low areas, which prevent the cutting of scour channels across the floodplain. In other cases, further scour can be prevented by the use of grass waterways across the flood plain.

Roadside erosion can be effectively controlled by properly sloping cuts and fills and then seeding to grass, vines, or shrubs.

There are other types of sediment control measures designed for specific purposes. Desilting basins offer immediate protection in cases in which a number of years may be required to install erosion-control measures on the land. Vegetative screens may also sometimes act as effective sediment traps at the upper ends of reservoirs.

Numerous studies have now proved that a complete soil and water conservation program, which treats all of the sediment sources involved, can be expected to reduce downstream rates of sediment production by 50 to 90 percent. This is possible without changing materially the basic agricultural pattern, and with the additional benefit of holding the soil on the land for higher crop yields. For example, the rate of sediment production to Jones Creek Reservoir in western Iowa has been reduced 88 percent by contour cultivation, retiring some cultivated land to pasture and woods, and installing gully-stabilizing structures.

In the case of channel sedimentation, however, it is usually not necessary to stop the sediment sources completely in order to stop the channel filling. Often if the sources of sediment are reduced by a relatively small amount, it will make the difference between a sediment load which the stream cannot carry and one which it can easily carry. The stream may then cease aggrading its channel and begin deepening it.

APPLICATION OF SEDIMENTATION DATA TO WATER PROJECT DESIGN

BY N. T. VEATCH*

It will be the purpose of this paper, so far as it can be done within the time allotted, to discuss the relationships between soil erosion and the design of reservoirs for use as water supplies. The subject is one on which no definite statements as to fixed formulae seem possible, due to the large number of variables involved and to the fact that the use of a formula necessitates the definite valuation of these variables. Several formulae have been suggested, but they have been developed from specific data obtained on certain projects or laboratory experiments, and there has been no acceptance of any formula in the same sense that hydraulic formulae have been used. For instance, there are three distinct types of artificial storage reservoirs used in water supply development. These are:

(1) Reservoirs created by dams in streams, where entire storage is within the banks of the stream.

(2) Reservoirs created by dams in streams where storage is provided by utilizing all of the stream bed and a part of the adjacent stream valley.

(3) Reservoirs created by dams, usually on small tributary streams, where at least a major part, and sometimes practically all of the runoff from the drainage area, is impounded.

I do not know of a good example of a water supply in Illinois corresponding to Type (1), but the Decatur supply would represent what was intended by Type (2), and the supplies at Bloomington and Springfield would probably be good examples of Type (3).

The use of sedimentation data in the design of a water supply would differ greatly for the three types mentioned. In the case of Type (1) unless a very unusual situation exists, siltation can be ignored, as the sediment that does accumulate in the reservoir, aside from a rather small area immediately above the dam, is washed out at each flood stage of the stream, and the original capacity restored. The design of a reservoir similar to Type (2) should include consideration of all pertinent sedimentation data. While in reservoirs of this type, usually much of the silt load reaching the reservoir during flood stages is carried through it, often the increase in cross-sectional area is such that a large percentage will be deposited. The silt carrying capacity of a stream depends upon the velocity, so it is clear that in any reservoir where the velocity is checked materially, there will be an accumulation of silt.

The effect of varying velocities differ widely, depending upon the type of silt load being carried, and therefore the amount of siltation would vary accordingly. It is probable that the effect of siltation, so far as capacity goes, is greatest in Type (2). This is true in spite of the fact that a greater percentage of the total silt load is retained in reservoirs of Type (3). The reason for this is that in reservoirs of the latter type, the silt is deposited in deltas at the upper end of the reservoir where the actual reduction in volume is small even though the area affected may seem large. In such cases there is some compensation due to reduced losses from evaporation.

Aside from the variables created by the type of reservoir involved, the character of the contributing drainage area, i.e., its slopes, amount and type of cultivation, the relationship of drainage area to reservoir storage, etc., all constitute reasons why, so far at least, it has been impossible to develop any fixed basis for the use of existing sedimentation data. The fact that almost invariably each supply presents an individual problem as the various factors such as those mentioned differ in each case, makes the development of any fixed relationship impossible.

It is not intended to infer that sedimentation data are of no value, and that they should not be given careful attention in the design of water supply reservoirs. In fact the opposite is the case. It is desired, however, to point out the many variables that exist and to urge that careful and intelligent use be made of existing data, in order to avoid reaching conclusions that are not practical.

The data on sedimentation that are now available are not, generally speaking, as extensive as would be desired, but there is quite an array of statistical information which when used with care and applied intelligently, can be of great assistance to the designer. In the design of reservoirs for water supply the following are some of the basic questions that should be answered:

(1) Is siltation a problem in the particular case under consideration?

(2) If so, what, if any, are the alternate locations available for development?

(3) Are there other sources more feasible or reliable than a reservoir?

(4) If a reservoir is the most feasible sup-

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ply and siltation is a problem, what should be done about it, i. e., provide additional reservoir capacity, use preventive measures on the watershed, or both?

(5) What capacity to allow for siltation, and its probable cost.

(6) What would be the cost of acquiring the entire watershed, and the cost of construction and maintenance of proven soil erosion preventive measures on it?

(7) What are the best preventive measures to use?

(8) If other sources of supply are available, is the reservoir project economically sound?

Of the above questions, those requiring the determination of whether or not siltation is to be a problem, the probable amount and what additional reservoir capacity should be provided to offset it, are the ones where the use of all available sedimentation data is necessary. The available data will all be from existing projects most of which have different physical characteristics. The data will have to be applied to the proposed project with care and discretion, and manifestly, the accuracy of the answers will be influenced by the experience and sound judgment of the designer.

In some instances there will be only one source of supply, and possibly only one feasible site for the reservoir. In such a case the question of the amount of siltation, and what to do about it, becomes an economic one in which the cost of providing protection against the loss of reservoir capacity must be fully justified. If the contributing drainage area is large, there is probably very little that can be done, but as in the majority of cases where the watersheds are comparatively small, much can be done.

Most of the sedimentation data now available has been collected by the Soil Conservation section of the Department of Agriculture, and by state agencies interested in water resources and soil conservation. The Illinois State Water Survey Division is an outstanding example of the latter. These data are to be found in publications of the American Waterworks Association, the American Society of Civil Engineers, certain Farm Journals, and in a large number of reports which may be obtained upon request, but which do not have general distribution. There is a list of references attached to this paper, which may be helpful to those desiring the information.

The only approach to mass data regarding siltation in reservoirs is contained in an excellent article by C. B. Brown (Journal AWWA, Vol. 33, 1941, page 1026), in which annual losses in storage capacity are given for 63 reservoirs. These reservoirs have widely divergent characteristics and nation-wide distribution. Unfortunately, detailed information regarding the reservoirs and watersheds was not given, therefore the real significance of the data is only apparent when considered along with such additional information. Knowledge of detailed data regarding several of the reservoirs listed on Figure 5 of the above mentioned article, demonstrates that two major conclusions are justified, namely:

(a) Silt should be kept out of the streams, as far as possible, by the control of erosion on the watershed.

(b) The capacity of a reservoir should be as large as feasible in relation to the area of its watershed.

The first conclusion is obvious, and the second borne out by practice in the waterworks industry. Numerically speaking, most of the impounding reservoirs used for public water supplies, are located in the upper reaches of watersheds, and have storage capacities ranging from 200 to 400 acre feet per square mile of watershed. As a comparison, the Decatur Lake (an example of Type (2) reservoir) has only 21 acre feet of storage per square mile of drainage area. There is some question as to whether or not this lake should be even considered as an impounding reservoir. Perhaps some term such as "channel reservoirs" should be used where the storage is less than 100 acre feet per square mile of drainage area.

It has been realized for some time that the logical approach to some of the problems of impounded supplies is through the care of the watershed. One of these problems is pollution and another is siltation. Within certain limits, pollution may be reduced by treatment, but practically nothing feasible can be done about silt after it has been deposited on the reservoir bottom. Pollution has been controlled by suitable regulations properly enforced, the water yield has been stabilized by suitable land use and, in some cases, works for the control of excessive soil erosion have been built. There are a number of outstanding examples of watershed maintenance, but these are limited to projects where all, or practically all, of the tributary watersheds are owned or controlled and therefore are not typical.

In the majority of instances where impounding reservoirs are used for public water supply, the purchase of land has been limited to that which may be inundated and the watershed land has been left in private ownership. Many individual farms as well as many individual owners are commonly involved and, in the absence of cooperation among these land owners, proper care of the watershed, from the standpoint of water supply, is hardly to be expected. It seems probable that the program of soil conservation which is now being carried out by federal, state and local authorities may produce the answer to the problem of multiple land ownership through the organization of the owners of watershed land and convince them that the primary benefits of soil conservation accrue to the land itself.

The progress of a soil conservation program is worthy of mention. All of the states have legalized the formation of soil conservation districts and a majority of the farm land in the country is now included in these districts. In view of such widespread participation in this program it seems reasonable to expect that the land owners will be organized on an increasingly large number of watersheds in the future and that the land will be improved and be made more favorable for reservoir operation. A pertinent point is that the organization of the land owners on a watershed may enable the reservoir owner to deal with one rather than many parties when considering needed control measures.

My personal contact with soil erosion is perhaps rather typical from the land owner's viewpoint. I have a farm located near, and draining into a small recreational reservoir in northeastern Kansas. The farm covers about 10% of the watershed and, prior to 1945, the greater part of it had been under continuous cultivation for many years without much consideration of modern methods of farming. Evidences of erosion were plentiful and I was advised that the land was better adapted to pasture than to field crops. At what to me was considerable expense, the land was terraced, planted to grass and steps were taken to control erosion at points where it was most noticeable. I now feed cattle, and am convinced that my investment in soil conservation was sound. Since the erosion prevention improvements were made, quite a few neighboring land owners have carried out similar improvements, and the benefits to the reservoir are quite noticeable. I decided to improve my land entirely from a selfish standpoint. It is believed that many other reservoirs can be benefited if the owners of farm land can be made to realize the value of soil conservation.

If I seem to be discussing agriculture rather than engineering, my excuse is that we must not lose sight of the fact that the control of soil erosion means more to the land itself than to anything else. When a supply of water is involved it may be expected to receive incidental benefits, but at the present time at least, the amount of land that is, or is likely to be, tributary to an impounding reservoir is small as compared to the total in agricultural use. We must remember that, so far as public water supply is concerned we can, in many cases, build new reservoirs when the old ones are full of silt since only dollars are involved, but we cannot replace topsoil without going through a long and expensive process of rebuilding fertility. A soil conservation program is a "must" in this country as the very existence of our race depends upon it.

Claims have been made that erosion control will reduce the cost of water treatment. It has been stated, that a saving of $7.00 per million gallons in the cost of water treatment "for all of the towns in the North Carolina Piedmont that have impounding reservoirs" would follow a reduction of thirty per cent in "suspended load of the streams." The statement is based on the reduction in the amount of alum needed and "on other savings resulting from smaller capital outlay for settling basins and filter plants."

We know that the need for coagulant is influenced by suspended matter and we can agree that the control of erosion on the watershed could effect a reduction of thirty per cent in the suspended load of the stream, but there is some question whether or not the suspended load of the effluent of an impounding reservoir would be affected to that extent by work on the watershed. Similar reasoning applies to the size of the basins and filters needed since most of the "suspended load" of the stream will be deposited as silt on the reservoir bottom. It seems probable that the usage of coagulant could be reduced during a part of the time, but the treatment plant, which must be designed to meet the worst of conditions, could not be changed materially, particularly when consideration is given to the added problems of treatment that may result from algae growths, micro-organisms, etc.

In the absence of control over watershed land and in view of the fact that funds for upstream engineering are likely to be either limited or unavailable, the engineer is faced with the necessity of designing an impounding reservoir for as long a period of usefulness as is possible under the existing conditions, which must include the ability of the owner to finance, as well as hydrologic factors. In spite of a number of instances of excessive silting, I believe it is possible to so design a reservoir that its useful life will be long enough to justify its construction. The most important feature of design is to make the capacity of the reservoir in relation to its catchment area as large as possible so that most of the silting will take place in the shallow waters at the upper end.

Mention should be made of the fact that the work of the soil conservation organizations is emphasizing the importance of the "capacity-to-area" relation by furnishing concrete evidence of actual silting rates. We have not had such detailed data in the past and therefore this is a distinct contribution.

It is certain that the soil conservation agencies should be complimented on the fact that their publicity has been in readable form, as compared to the bulk of articles on sedimentation. A certain amount of precise nomenclature is essential but it seems as if many of the terms used in articles on sedimentation could be stated in plainer English and with resulting advantage to everyone.

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WATERSHED APPROACH TO SEDIMENTATION PROBLEMS

BY ORVILLE W. CHINN*

The recent devastating floods in the Midwest with their accompanying soil erosion and sedimentation damages have again focused attention on the need for complete watershed and stream basin protection program.

Starting with the watersheds in their present condition we can hardly expect to make a perfect job of flood control and sediment damage prevention. It is, however, within the realm of probability that we may be able to materially reduce these damages.

We can at the present time, also by looking about us, profit by previous mistakes and begin a program that will ultimately lead to a high degree of water and sediment control.

Our sediment problems start at the point the rain drops first strike the earth, blasting the soil particles out of their positions like small bomb explosions, and the problems stay with us all the way down the hillsides, through the tributaries and along the main stream channels until they reach the sea.

Contrary to a common belief, the result of the damage from these displaced particles is proportionally greater in the upper reaches of the watershed than in the lower. It is estimated that the topsoil off one 40-acre farm passes Vicksburg each minute, but there is no estimate of how much topsoil is moved about from its original location in our fields down the hillsides on to lower areas or to the small reservoirs and tributary stream channels in our headwater areas.

In our State in particular a considerable portion of flood damages could be charged directly to this sedimentation of our small stream channels.

In approaching the solution to our watershed problems we should be as practical and as realistic as we know how, and should start a coordinated attack at the source of the trouble and should progress downstream as rapidly as possible.

On small watersheds, solutions are often simple, but on the larger watersheds with a multitude of land owners, railroads, highways, State and Federal lands, the solution is much more difficult.

The Committees on The President's Water Resources Policy Commission were practically unanimous in their recommendation, that a comprehensive program for the development of both water and land resources should be carried on. Just how such a program should be initiated and carried out was never completely decided, as there appeared to be as many opinions as there were committees working on the commission.

It was generally agreed that the Federal government should accept the responsibility for safeguarding and developing our natural resources. The commission's opinion also was that planning should be carried on at a national level, rather than at a watershed, state or regional level.

What our approach on a watershed basis should be is quite a problem, and we have seen many partial solutions during the past two decades.

Among those projects we have observed on a major watershed basis is the Tennessee Valley Authority which has done a wonderful job of developing the water resources of the major streams in the Tennessee Valley primarily for flood control, navigation, and power, and secondarily for recreation, water supply, and industrial use. Located in a region that is nearly half forested, sedimentation of the TVA reservoirs is not a major problem. The useful lives of these reservoirs are estimated by TVA engineers to be from several hundred to several thousand years. Public forests in the Valley have good management including fire protection. On the privately owned forest lands, which comprise more than four-fifths of the total forest area, substantial progress has been made in watershed protection since TVA was created in 1933 with consequent decreased erosion largely as a result of improved fire protection by state forestry and county organizations stimulated and encouraged by TVA. Improved forest management developed through TVA cooperation is also a factor. CCC camps working throughout the Tennessee Valley did effective work on hundreds of acres of land in erosion control through tree plants and gully control.

With respect to the open lands in the Tennessee Valley, the problem of erosion control, except for the CCC work, has been handled as a part of the agricultural program administered by the agricultural colleges in the Valley states. The methods used have been educational and include test demonstration farms scattered throughout the area. In 1949 there were 6,000 of these test demonstration

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farms in the Valley. Phosphate and other fertilizers have also been distributed widely over the area on a demonstrational basis. While this program has undoubtedly accomplished some results, it has not proven to be the answer to the erosion control problems of the agricultural lands in the Valley, although it has been in operation since 1933. This would seem to indicate that educational and demonstration methods alone will not do the job even when concentrated in one basin.

Other than the TVA, the Federal agencies interested in Valley development are the Corps of Engineers, the Bureau of Reclamation and the Department of Agriculture. The first two mentioned agencies are primarily interested in water development, and have not devoted much time to the problems of sedimentation control except to study the effect of sediment on the reduction of reservoir capacities and maintenance of navigation channels.

In the Department of Agriculture, the Forest Service and the Soil Conservation Service are the two bureaus most vitally interested in complete watershed protection and flood control and sediment reduction.

The Soil Conservation Service working under the authority of Public Law 46 carries out a program of soil conservation through organized Soil Conservation Districts. This program is based on the principle of using every acre of land according to its capabilities and treating it according to its needs.

The land treatment measures, including proper land use, reforestation, improved pastures and meadows, gully control, terraces and diversion ditches on croplands if properly applied and maintained reduce erosion, increase infiltration and reduce the sediment loads of our creeks and rivers.

This type of treatment if concentrated in a small watershed where a high percentage of application is obtained is effective in reducing sedimentation as may be shown by the following examples. In the Newman-Georgia reservoir the rate of siltation from a 1.39 square mile drainage area was reduced from 0.5% of the lake's capacity per year to 0.15% per year by installation of conservation measures. This was a reduction of 78% in the rate of silting. Siltation was reduced from 2.2 acre feet per square mile of drainage area during the period 1938-41 to 1.02 acre feet per square mile during the period 1941-49 in Lake Issaqueena in Pickens County, South Carolina. This is an annual reduction of 53% in the siltation rate, and was accomplished principally by changes in land use and treatment. Many other examples may be cited supporting this type of treatment.

The problem confronting us at present is how we can get an effective coverage of conservation practices established in a watershed.

The present Soil Conservation program as now operating, gives a too widespread and scattered an application to be effective. Using Kentucky as an example, we find that over a period of ten years, 12-1/2 percent of the farms in the state have soil conservation plans. Of these no more than 40% of the recommended water control practices have been applied (that's a generous estimate), which leaves us with 5% of the needed practices applied to date. This is too thin a coverage to make any appreciable reduction in the siltation of our streams. In fact, the rate of silt deposition in the small streams has increased tremendously within the past eight years, particularly in the smaller streams.

The Muskingham conservancy district in Ohio, which is a local organization, working closely with the Soil Conservation Service and other Federal agencies, and using their own technicians full time in the Muskingham watershed have substantially reduced the siltation rate in their reservoirs. In that connection they control more land. They were able to buy up all the farms. This would seem to indicate that an active interested local organization can obtain results with the present agencies on a limited scale.

The Missouri Basin comprehensive plan on the other hand has so many agencies involved and such a multitude of generalities, plans and details which cover everything from major stream improvements to gopher eradication, and teaching farm people how to use electricity, that it appears it may never produce any action or benefits to speak of, and may eventually collapse of its own weight.

We can keep planning and planning, and there is no end if we keep on planning.

The Department of Agriculture Flood Control program as authorized in 1936 could be a highly effective weapon in combating headwater floods, erosion, and siltation. The program includes all the conservation measures recommended on private lands, highways, and railroad rights-of-way within a given watershed. In addition, upstream reservoirs, desilting basins, channel improvement, and bank stabilization are provided for. This program provides a complete supplement to the work that the Corps of Engineers are doing on the main waterways for flood control and navigation. The program provides for a concentration of conservation and water control measures in a specific watershed. The larger watersheds are divided into minor watersheds and plans for complete coverage of practices are set up with the higher priorities given to the more critical areas. Operations in the minor watersheds are carried on through organized Soil Conservation Districts. The individual farmers aided by Department technicians initiate the conservation practices on their land. The larger operations where group benefits are involved are planned by the Department technicians and construction or

installation is done by private contractors at Federal expense.

The combined operations of the Department of Agriculture and the Corps of Engineers, together with the States involved, will give a drainage basin adequate protection from floods and siltation. The two programs should be planned and carried out simultaneously, or if at all possible, the agricultural program should be carried on ahead of the engineering program. This unfortunately has not been done in the past, and the Department of Agriculture has only eleven authorized flood control projects in the United States where hundreds are needed.

The principal weakness of this coordinated program has been the time lag between the field surveys to determine the need for the operations in a watershed and the time such operations actually commence. In many instances this has been ten or more years. Such delays are not easily justified in the eyes of a farmer who sees his farm land covered with swirling muddy water several times a year. He cannot understand how changes in policies and procedures at top levels in Washington can delay so necessary a program for so long a time.

In summation it may be said that to be effective and practical a flood control and sediment control program for a watershed must be financed to a large extent by Federal funds. It should include all necessary conservation and engineering works from the watershed to the mouth of the stream. It should be planned and administered by the minimum number of Federal agencies possible. It should be planned and carried out in cooperation with the state or states affected. If it is to be effective during our lifetime, and while there is still time for a remedial program to be effective, Congress or cabinet authority should see to it that such a program can clear all necessary channels and be ready for presentation to Congress in no more than six months from the completion dates of plans or survey reports.

WATERSHED APPROACH TO SEDIMENTATION PROBLEMS

BY WENDELL R. LADUE*

WITH DISCUSSIONS BY H. A. EINSTEIN AND ALEX VAN PRAAG, JR.

It has been said that good water and land management go hand-in-hand. Water supply engineers, foresters and land-use planners are learning more and more that the relationship between water supply and the condition of land resources is intimate and continuous. The whole purpose of watershed management is to maintain this relationship so that all watershed resources may be utilized to their fullest advantage and extent.

As an example let us take a specific watershed, Fig. 29, the watershed which contributes to the Akron, Ohio water supply. The watershed area is 207 square miles, is entirely glacial in origin, and the average elevation is about 700 feet above Lake Erie. The normal rainfall over the watershed is 20 per cent greater than at Cleveland, and about 15 per cent greater than at Akron.

In fact, the Cuyahoga River itself is of glacial origin. The source of the Cuyahoga River is about 20 miles from Lake Erie, thence the river flows southerly to Akron, then reverses itself, entering Lake Erie at Cleveland about 30 miles west of the source. The upper reaches of the river—with 150 square miles watershed—descend through flat glacial lands. Many small lakes and man-made ponds abound in this area. From this point (a), Fig. 29, southerly to about 2 miles below the Lake Rockwell intake the river is rather sluggish, with a drop of about one foot per mile to the reservoir (b). Downstream are rapids composed of rocky areas exposed in the bed of the stream. Within about five miles the drop is 25 feet. Through Cuyahoga Falls (c) the descent is rapid with a drop of 160 feet in three miles. From there (d) on to Cleveland the stream follows a deep-filled preglacial valley with precipitous slopes but even beds.

Let us see just what has been done both by nature and by man. We have a reservoir at Lake Rockwell which was filled, in 1914, to a capacity of 7,423 acre feet. In the 36 years there has been a loss of only 539 acre feet or seven and one-quarter per cent, equivalent to one-fifth of one per cent per year. This means there will be no reservoir after 500 years. Some may ask: is 500 years enough? What has brought about this control? We have to consider nature and man aiding each other to control erosion.

What has man done? He has entered the picture and developed certain procedures for reducing silt production to help nature. I might say that nature is very kind if man will only let her help.

It is of interest to note here that the watershed manager must be a person of many parts. He must be a meteorologist, to know his rainfall and runoff records. He must be a chemist and mineralogist because he must know about the chemicals and minerals in the area. He must be a bacteriologist if he is to recognize water-borne diseases, and a biologist if he will recognize and classify the plant and animal life. He must be a forester in order that he may restore forests and that he may also know how to profit by existing forest cover.

Starting in 1922 we have planted some 800,000 trees about Lake Rockwell. On this area we have

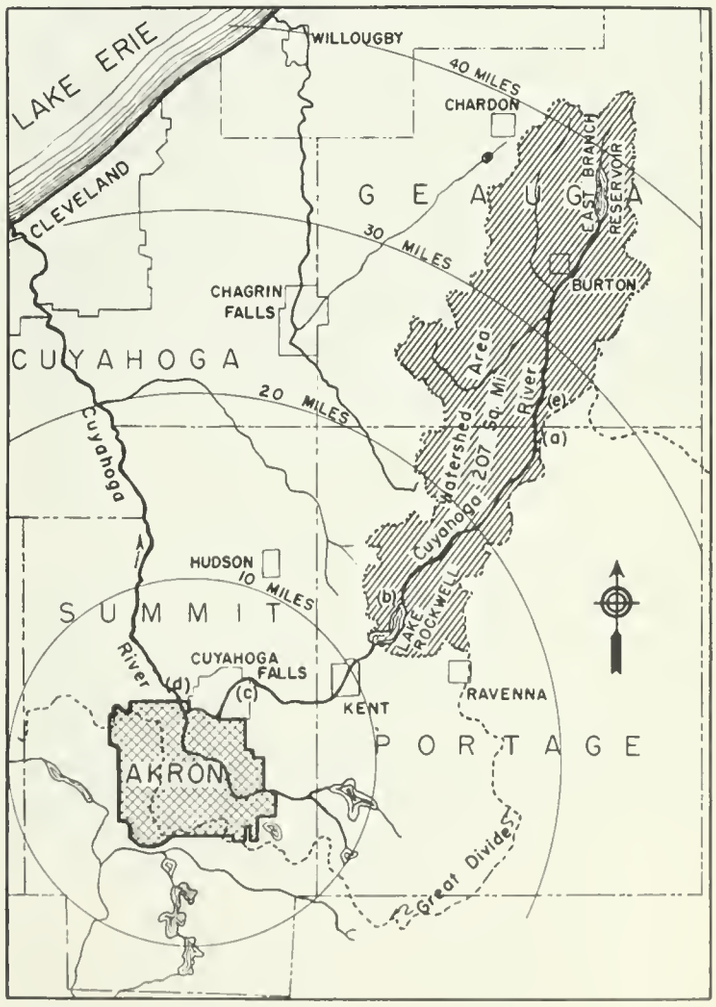

FIG. 29.—CUYAHOGA RIVER WATERSHED AREA ABOVE AKRON, OHIO.

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learned to be forest experts. We go through our forest every year and cull out what is not good. The city owns 8,000 acres scattered all over the watershed for a distance of 15 miles above Hiram Rapids (e). On those areas we have farms, 250 cattle, a thousand sheep and a few hogs. He must be an expert in animal husbandry and sometimes he must be a veterinarian. He has to be a farmer. This fall we harvested 18,000 bales of hay, 6,000 bushels of oats, likewise corn and wheat. At the present time we are getting ready to sell cattle. In this business of operating so much land all types of farming operations will be experienced. We have 150 acres of fruit trees from which we harvest 35,000 bushels of apples.

He must be a geologist. We have coal, sandstone and shale formation 700 to 800 feet, which have to be recognized and evaluated. He must be a soil mechanic in order to build dams with what he has to work with. Eventually and inevitably he will become a conservationist, interested in the planting of trees and the conservation of soil. Above all, he must be an hydrologist, dealing with one of nature's phenomena. It seems strange when we say that we are a part of all these things, and that a waterworks engineer should digress so far. But, if he is to protect his watershed and the future of his city and community he should at least know his area like a book in all its chapters. The problems involved in its efficient management require his personal interest.

I would say that the greatest advantage derived from doing all these things is that we have become good neighbors. We own only 10 per cent of this watershed. We have set an example. We find other people copying what we do. Therefore over the past few years the watershed has changed considerably. I must direct your attention to the fact that the watershed is only about 30 miles from Cleveland, 30 miles from Youngstown and 30 miles from Akron—right in the center of the triangle with a large city at each apex and with almost one-third of the entire population of the state of Ohio within a 60-mile radius.

It is to be noted that in these pursuits we deal with continually changing unknowns—rainfall, runoff, forest cover, cultivated lands, crop rotation—all changing to the vagaries of both man and nature. Each watershed has a personality of its own—to be studied, surveyed, watched, directed, controlled. Here at least on our watershed we have a 36-year record of what has occurred. With continual watershed knowledge a pattern is set for what to expect. But, in all, it figures intimate, prolonged and watchful acceptance of the relationship existing between man on the one hand and water and land resources on the other. Is 500 years enough? Have we a yardstick? At least we have a yardstick for one area of the United States. Others will follow for comparison. Here is a true approach to a sedimentation problem. We have an answer, worked back to the original question—what caused such low sediment conditions? In no other waterworks structure devised by man is the useful life susceptible to greater prolongation by the simple exercise of preventive maintenance. I would predict that all designers and builders of reservoirs in the future will include in their plans items of siltation control and watershed management "upstream engineering" resulting in the immensely profitable and measurable item of prolonged useful life to the structure. In this nature will help—if man is willing.

DISCUSSION

H. A. EINSTEIN. —We have been shown very convincingly by the two speakers how important soil conservation measures on the land are, as far as the downstream runoff of sediment is concerned. But there are a few points that complicate matters.

One becomes apparent in the Oxford (Mississippi) area of which we have seen pictures. Numerous large gullies have developed there in the sandy soils of the watershed. The difficulty is that one probably cannot convince any land owner that he can improve the condition of his land by work on the gullied areas. There is a big danger that those people will work on their best areas only and leave the poorest areas to the elements. This is a big source of sediment that will not be touched by normal conservation methods. It must be paid for by the people that will benefit downstream, and that is a difficult problem of organization.

The second point is, that even if someone would stop all those gullies today such that no sediment would leave the area, it would probably take 20 or at least 10 years until one would feel a reduced output of sediment at the lower end of the 125 square mile watershed. In other words, there is so much sand accumulated in the channel system it will take that long to clean it out and only then will the sediment load at the lower end be reduced. If a reservoir were built at the lower end of the watershed it would accumulate all the sediment which is stored in the channel system at the time.

That ties back to what was said about the division of the sediment load. The silts and clays, which are not represented in the bed, move through the system at high speed. Any change of silt output in the watershed will be felt immediately downstream. The material that moves as bed-load on the other hand moves very slowly and is accumulated in the system and, therefore, its motion downstream is slowly affected by changes upstream, often only after many years.

There is one third point. Sometimes a definite possibility exists of improving the sediment problem in the reservoir without having to go all the way to the watershed. This may be done by providing sediment deposition space near the reservoir in the valley of the sediment-carrying stream. Nature has provided for this possibility, for instance, in the Rio Grande valley above Elephant Butte. Today, a large percentage of the sand load is deposited in the valley above the reservoir. If I remember the figures correctly only ten per cent of the bed-load which moves in the Rio Grande at San Acacia, about 40 miles above the reservoir, actually reaches the reservoir. All the rest of the bed-load and much of the silt is deposited in a very dense vegetative screen of tamarisk which has developed in that part of the valley. Actually the possibility exists of providing artificially such a screen upstream of many reservoirs in order to protect the reservoirs themselves. This provides a possibility of depositing both the bed-load and at least part of the fine material where it causes a minimum of damage.

ALEX VAN PRAAG, JR. —I am quite a little interested in the subject myself, coming from Decatur, which has a lake created by the damming of the Sangamon River. The watershed area is 900 square miles and it is 90 per cent in cultivation. There is very little noncultivable land.

We have a very serious sedimentation problem. In our situation, in contrast with the fine experience which has been recited, the city of Decatur thus far has only talked about what it ought to do. The exception to this statement is that the city has hired a full-time conservationist to work with the land owners in effecting a watershed land conservation program. It financed his services out of water department funds. Can you imagine, though, how little one man alone can do?

How seriously does this affect our water supply? Our lake has been one of those studied, and by very conservative prediction we can be without sufficient water in 1954. One-third of the lake capacity has been lost to silt deposit. We continue to expand our water uses, and should we suffer an extreme drought at any time, we can be in a very bad shape. Yet, our citizens go out and see this lake and they think, "with that lake for a water supply, we can never be in trouble."

ANALYSIS AND USE OF GROUND-WATER DATA

BY W. F. GUYTON*

WITH DISCUSSIONS BY JOHN H. BLISS, HOWARD CRITCHLOW AND H. A. SPAFFORD

It gives me a great deal of pleasure to be here today and to take part in this meeting honoring the Illinois State Water Survey Division. I have known of the work being done on ground water by this Division and by its parallel Geological Survey Division for a number of years, and have always had reason to have the highest respect for these two Divisions.

As you probably know, the State of Illinois is somewhat unique in that it handles most of its ground-water investigations alone, without the financial cooperation and assistance of the United States Geological Survey. It has held its own very well in comparison to the other states, even without the help of the national organization, and is to be highly complimented for the good work it has been doing.

During recent years, national attention has been drawn to ground-water problems at an increasing rate. Probably you have read some of the articles describing the ground-water situation throughout the country. Some of these articles are quite factual and present the story just as it truly is. Other articles, however, have been written by persons who were not thoroughly familiar with water resources problems but who were motivated by a desire to produce something sensational. They have cried that the ground-water resources of the United States are decreasing and that serious consequences will result before many years pass. The truth is, though, as has been pointed out by a good many people in a position to know, that the supplies are not decreasing; the water level is decreasing, and there is no such thing as a national water shortage. Our surface-water and ground-water resources are not running out. There is just as much water today as there was a good many years ago. As you know, water is a renewable resource, and it is replenished year after year by what is known as the "hydrologic cycle."

The real trouble is simply that people are using more and more water and costs are going up as a result. In 1945, I made a survey of ground-water use and found it to be about 20 billion gallons per day on an average. In 1935, Dr. O. E. Meinzer, of the U. S. Geological Survey, had estimated it to be about 10 billion gallons per day. In 1950, K. A. MacKichan of the Geological Survey estimated that the use of groundwater was 30 billion gallons per day. Thus, in 15 years the use of groundwater has tripled.

Now, it is axiomatic that every time a second or a third user of groundwater puts his well into a ground-water reservoir and starts using the water, it lowers the water level in the first well and all other wells that have been drawing from the reservoir before the new well was put down. In order for groundwater to flow to a well, it must have a gradient, and that gradient must be created by lowering the water level in the well. Such lowering of the water level in the well that is pumped also lowers the water levels in all the other wells in that reservoir between the pumped well and the areas of natural recharge and discharge, and also for some distance laterally on either side of the pumped well.

Thus as more and more ground water has been used, and new wells have been installed, the water levels have been lowered more and more, and some people have gotten the idea that ground water is running out. Where there is sufficient recharge to supply the withdrawal, however, this lowering of water levels does not indicate overpumping. It is simply a matter of diverting the recharge to the wells for use by man rather than letting it flow to waste. It is true that if there is not enough recharge to supply the withdrawal, the water levels do keep going down and down and the ground-water supply eventually is exhausted, but fortunately, in most of the country there is sufficient recharge and the problems are largely problems of economics with respect to pumping lifts or distribution of wells rather than being problems of actual shortages of water.

Of course, there are some real shortages of groundwater in the United States, and I do not want to give the impression that there are not real and serious problems. I have worked on some of them myself. In parts of California, Arizona, and New Mexico, and in the High Plains of Texas and some other areas, there simply is not enough recharge to supply the demands being made on the wells. In those places the groundwater is being mined, and eventually Old Mother Nature will force a cessation of the overpumping because the water levels will be drawn down so far that the cost of pumping will be exorbitant or the yields of wells will be decreased to such an extent that the large amounts of water

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can no longer be obtained.

In one area where I worked last spring, near Deming, New Mexico, we made a special effort to point this out to the local people. We told them that they are using the water at a greater rate than the recharge and that, while they are making money now by irrigating, there will come a day when they will not be able to make money in this way because there will not be enough water left, and that as a business proposition they should plan their developments toward that eventuality.

Often, people feel that legal control of ground water is the final answer; but legal control, I am afraid, cannot do the whole job. There are problems such as those in New Mexico and the High Plains of Texas where the recharge is almost negligible in comparison with the amount of water being pumped, and the people simply will not stand for regulating the use of water to the amount that is available by natural recharge. They want to use this water the same as they would use coal from a coal field. And since, in most cases, they have already started using water at excessive rates before legal control is proposed, it is practically impossible for legal control to do more than to hold the problems to a minimum after that time. The answers there still are a long way off.

In many other places, we are just now on the threshold of making real use of our ground-water resources, and legal control might well be a hindrance to the development unless very carefully applied. More often than not there just is not enough information available now to make and apply laws that are just to everyone. Too strict legal control could well stifle progress, whereas too weak legal control would be a useless burden on the taxpayers.

One thing is certain, whatever is done, the persons charged with regulating our ground-water resources must be truly wise and honest men, and must be given a wide latitude for the use of personal judgment. The situation appears to be just too complicated to be handled successfully in any other way.

Fortunately, though, be that as it may, the over-all situation is not as bad as has been presented by some people. Though the total amount of water, both surface and ground water used in the country in 1950, as estimated by MacKichan, was 170 billion gallons per day, it has been estimated also by the Geological Survey that the amount of water available for use, or rather the amount of water flowing to the oceans from our streams, is approximately six or seven times this amount. Thus, on the whole, there still is several times as much water unused as is being used.

Ground-water reservoirs contain at one time or another a sizable part of this water as it flows, from the areas where the rains fall, back to the ocean. If we develop and use this presently unused water, both ground water and surface water, in the proper manner, there will be room for a great deal more expansion in the use of water in most places in the country.

The question of developing and using water in the proper manner is a big one. To do so we must know something about the source of the water, how much water there is in storage at the source, how much the recharge is and how it fluctuates, what the best methods of getting the water might be, and many other things that go along with development. All of this calls for what is called "basic data." Unfortunately, however, the basic data available on the water resources of the United States are rather skimpy in comparison to the needs for such data and the great programs of development that are being undertaken.

In many cases in the past, basic data have not been obtained for our water resources until the need has arisen for such data. Then, in many instances, when the need was there, it was too late to get all the data that were needed. Such things as record of stream flow, water levels in wells and pumpage from wells must be collected and compiled on a continuing basis. Once the water levels have fluctuated in wells it is too late to go back and find out what happened. So, in many cases where development has been made in the past, it has had to be made partly by guess, with only the use of data that could be obtained relatively quickly. The same will be true of developments that will be made in the future unless more attention is given to the collection of basic data now.

The Illinois State Water Survey Division and the United States Geological Survey, and the others, certainly recognize the need for these data, but the programs which they have proposed time and time again for collection of basic data have been trimmed by the legislative bodies in order to leave money for more immediate problems. In the opinion of those of us who have to deal with water resources problems, this is false economy.

It is hard enough to solve ground-water problems without wasting money, even when all the data we can think of are available. It is much, much harder when data are missing because they have not been collected over the years passed. Without data there must inevitably be fumbling and bad guesses which lead to improper design of wells, the wrong spacing of wells, overpumping of reservoirs, too much interference with existing developments and the like.

So, before I go further with this talk, I would like to make this plea: that the basic data programs proposed by the state organizations and by federal organizations, who collect the data, be given careful attention and be supported in every manner possible. The cost of such programs is only a drop in the bucket as compared to the waste of money that results from lack of enough data.

This title is comprehensive. Mr. Hudson suggested that I endeavor to review the way in which ground-water data are used so as to give the state agencies and others an idea as to what kind of data are needed. He also suggested that I briefly review the non-equilibrium theory of groundwater, because this theory is not too well known in many circles.

In regard to the ground-water data, I might first mention a few of the types of basic data which we need. The first class of data might be called "geologic." As you know, groundwater occurs in formations of sand or gravel, or in limestone or other types of rock beneath the land surface. These formations are sometimes regular in shape but are more often irregular in shape and composition. They have been laid down according to natural laws and under conditions that can be determined by proper study of the geology. A study of the geology of a water-bearing formation is about the same to a ground-water investigator as the building of the framework of a house is to a carpenter. We must have the information from the study of geology before proceeding to find out how much water there is in the formation and how to get it out in the best manner. From the geology and from test drilling and geophysical studies we learn the location and depth of the formations, their thickness, the types of materials comprising them, and the composition of those materials, and the inevitable changes and irregularities that occur in the formations.

The next most important data are water levels and pumpage records of wells that have been drawing from the formations. At least this is the case where there are wells because, as you know, hindsight is usually better than foresight.

In most formations the flow of groundwater is laminar. In other words, the rate of flow is directly proportional to the hydraulic gradient. Also, in determining the drawdown of water level in a formation, generally the effect on the water level caused by pumping one well can be added to the effect caused by pumping another well in order to determine the combined effect of both wells. This makes the solution of a ground-water problem much simpler than it otherwise would be, because we can break the problem down into small components and then add it all up again and get the answer. If we know what has happened in a formation over a period of years as a result of pumping from that formation, a detailed study of the past records, taking them apart and then putting them back together again, will give a mighty good indication of what will happen in the formation in the future if the pumping is increased or if pumping from another part of the formation is commenced.

Other important data are those obtained with respect to recharge and natural discharge of the formations. Just where are the areas of recharge and how much water can enter the formations under various conditions? Also, what is the chemical character of the water in different places in the formation, and its temperature? To many users the characteristics of quality are more important than those regarding quantity. Quality records have also quite a place in determining the quantity of water available at different places, because they give a clue as to where the water is coming from and how much can flow from one area to another.

Then, of course, there are the usual data with respect to yields and methods of construction of wells, and the various problems that go along with getting the most water out of a well.

I have left a very important set of data for special mention. These data deal with the ability of the formation to transmit water from one place to another, and its ability or capacity for storage of water. As you probably know, ground-water reservoirs act both as conduits and as storage reservoirs. We call them reservoirs, formations, aquifers, or what have you, but they all store water at least to some degree; and if the water is any good, it must be flushing itself out from one place to another in the formation, however slowly.

The characteristics of a formation to transmit water may be called its permeability, or its transmissibility. Permeability generally refers to the material and transmissibility to the formation as a whole. The ability of the formation to store water or to hold water in the interstices of the sand and gravel or the cavities and crevices of limestone, or other rocks, may be catalogued by its specific yield or effective porosity or storage coefficient. These terms indicate how much water can be obtained from the formation by lowering the water table. The storage coefficient is also applied to artesian conditions to indicate how much water is obtained from the formation under artesian conditions as a result of compression of the formation and from leakage through confining and from included fine-grained beds, when the artesian water level is lowered.

These data may be determined in part by laboratory tests, but it generally is found that pumping tests of wells, both individually and in groups, can be used to determine the characteristic more accurately and on a more general basis for the formation as a whole. Such pumping tests have been developed rather extensively in connection with the non-equilibrium theory which I expect to go into a little later.

In regard to all these various data, before discussing the non-equilibrium theory, I would like to say I have yet to run across a single consulting problem in groundwater where at least some of the data were not needed and where I did not have to call on the basic data agencies for help in this regard; or, if they did not have the data, to go out and

obtain it myself.

The solution of a ground-water problem generally is like an equation 1, 2, 3. Rarely are enough data available so that it is just one, two, three, or so that one line of reasoning is enough. You go at the problem from several angles, each one of which will give you an idea as to what the answer is, but all the pieces must fit and make sense and agree with one another before you are sure that the right answer is obtained. So we use all the data that are available—water levels, pumpage records, records of yields of wells, transmissibility and storage coefficients, chemical analyses, temperatures, data on recharge and discharge areas—put it with what we know about the geology, the structure and position and formations, and see if everything adds up to one answer. If not, there is doubt as to whether the right answer has been obtained. Where too much data are missing, it is like groping in the dark for a light switch. I have a witch stick. I could demonstrate its use to you here, but that does not mean that I, or anyone else, can go out and witch water. We must learn the answers from a study of facts, analyzed in a scientific manner; and these facts which are to be analyzed include the basic data that are collected by the data-collecting agencies.

Now, to go on briefly with the non-equilibrium theory. This is really not just a theory. It is a concept based on natural laws of occurrence of groundwater which have been discovered and proven to be correct over a period of many years. A study of this concept, the formulas which have been derived, and the coefficients which have been identified, has shown it to be a fundamental concept of physics that is applied not only to groundwater, but to electricity and to the flow of heat and to many other problems. While it was being developed in the field of groundwater, it was also being developed in a parallel manner in the entirely separate field of petroleum.

This concept, or theory, has added a powerful new tool to the ground-water investigator's set. Its use, when enough data are available, puts the solution of most ground-water problems on a sound theoretical as well as experimental basis, rather than leaving a part of every problem to be solved by empirical means. Even if enough data are not available, the non-equilibrium concept is fundamental to a proper understanding of what might be the solution, and what lines the investigation should follow.

One of the things included in this concept is a recognition of artesian storage of water, i.e., water which is obtained from storage in the formation without any unwatering of the water-bearing material. Meinzer was the first to be given credit for the concept of compressibility and elasticity of artesian water-bearing formations. He studied the Dakota sandstone and found that, although the withdrawals from wells were not being balanced by recharge, the formation was not being de-watered. This showed that part of the water withdrawn either was coming from compression of the formation, or from leakage through confining layers, or that expansion of gas in the water remaining in the formation was replacing the water taken out. Since the composition of the confining beds and the water was such that it was not conceivable that all the water was coming from leakage, nor was it conceivable that the water remaining contained enough gas to take the place of the water withdrawn, all Meinzer concluded was that at least part of the water was coming from storage as a result of compression of the formation and from expansion of the water in the formation which accompanied decreased internal hydrostatic pressure. Meinzer's first statement on this was published in 1925.

In 1935, Theis published what has since become known as the Theis non-equilibrium formula. This formula takes into consideration the time element, and the withdrawal of water from storage, both artesian and non-artesian. Based on certain simplifying geologic assumptions, it gives the drawdown of the water level at any place in a formation that is caused by pumping from any other place for any given length of time.

In order to use this formula and to adjust computations made with it so that all variations from the assumed geologic conditions are taken into account, we must know quite a lot about the formation; and we must accept the axiom of balancing the ground-water recharge and discharge in the formation before equilibrium can be reached. Theis has explained this, as well as his formula, in about as good a manner as anyone can explain it, in order to save water, and I would like to quote from a paper by him, originally published in the Civil Engineering magazine for May, 1940:

"It is evident that on the average the rate of discharge from the aquifer during recent geologic time has been equal to the rate of input into it. Under natural conditions, therefore, previous to development by wells, aquifers are in a state of approximate dynamic equilibrium. Discharge by wells is thus a new discharge superimposed upon a previously stable system, and it must be balanced by an increase in the recharge of the aquifer, or by a decrease in the old natural discharge, or by loss of storage in the aquifer, or by a combination of these.

. . .

"Under Darcy's law there is only one way of reducing the flow in the areas of natural discharge or of increasing the flow in the areas of recharge. This is by changing the pressure gradient or the thickness of saturation of the aquifer in those areas, which in turn means changing the height to which water levels rise in wells throughout the area be-

tween the producing wells and the areas of natural recharge or discharge. This means a lowering of water level everywhere between the wells and the areas of natural discharge or recharge. In turn this means a reduction of storage in the aquifer and abstraction of water from it.

"There are two fundamental physical properties of any aquifer which largely control the movement of water through it. The first is the ease with which it transmits the water, analogous to the thermal conductivity of a solid in the theory of heat, or the electrical conductivity of an electrical circuit. This characteristic of the aquifer as a whole is called the coefficient of transmissibility.

"The other important characteristic of the aquifer is the amount of water that will be released from storage when the head in the aquifer falls. This has been called the coefficient of storage, and is defined as the amount of water in cubic feet that will be released from storage in each vertical column of the aquifer having a base 1 ft. square when the water level falls 1 ft. For non-artesian aquifers the coefficient of storage is nearly identical with the specific yield of the material of the aquifer. For artesian aquifers the coefficient depends on the compressibility of the aquifer or of included or stratigraphically adjacent shaly beds and is much smaller.

. . .

"When a well is drawn upon . . . water levels are drawn down in the vicinity of the well. Some water is removed from the vicinity concurrently with this reduction in water levels, and a so-called cone of depression is formed. The shape of this cone is determined principally by the ease with which water flows through the aquifer—the coefficient of transmissibility—and by the coefficient of storage.

". . . With continued pumping the cone deepens and broadens. It is evident that the well is taking water out of storage in the vicinity and that as more and more water is removed by the well, the cone of depression affects more and more distant parts of the aquifer.

. . .

"The formula for the cone of depression in the ideal homogeneous and isotropic aquifer assumed is

$$v = \frac{114.6\,F}{T} \int_{z}^{\infty} (e^{-u}/u)\,du\,,$$

in which

v = drawdown at any point, in ft.

F = rate of discharge of the well, in gal. per min.

T = coefficient of transmissibility

$z = 1.87\, r^2 s/Tt$

r = distance between pumped well and point of observation in ft.

s = coefficient of storage

t = time the well has been discharging, in days

u = a dimensionless quantity varying between the limits given.

. . .

". . . The important general principle is that, according to the formula, which appears to hold except for very short periods of pumping, the rate of growth and the lateral extent of the cones of depression are independent of the rate of pumping. If we pump twice as hard the cone will be twice as deep at any point, but it will not extend to any more distant areas. The disturbance in the aquifer created by the discharge of the well may be likened to a wave; the amplitude depends on the strength of the disturbance but the rate of propagation depends only on the medium in which the wave is formed. The reservoir from which the well takes water is almost as closely circumscribed by time as it would be by any material boundary, and until sufficient time has elapsed for the cone to reach the area of natural discharge and, or the area of rejected recharge, a new equilibrium in the aquifer cannot be established.

. . .

"After the cone of depression reaches areas of rejected recharge or natural discharge, it is modified by the effects of adding water in the former or preventing it from escaping in the latter. If the rate of pumping does not exceed the amount of water added in the recharge area and that prevented from escaping in the discharge area, the cone will eventually reach equilibrium, at least practically speaking. The approximate effects that occur after the cone has reached the boundaries of the aquifer can be estimated by means of various mathematical analyses. The effects of discontinuous pumping can also be evaluated." End of quote from Theis.

The development of the Theis non-equilibrium formula and its application has led to many pumping tests of formations to determine the transmissibility and storage coefficients. Along with these pumping tests there have been developed various techniques for analysis of the data and various applications of the formula to widely different boundary conditions. Many of these things have been written up and published in one place or another, but on the whole they are still in a state of flux. There is no single comprehensive manual on pumping tests nor is there a single comprehensive paper or book on the application of the results of pumping tests to all types of problems. Generally, each ground-water office has its so-called specialist on the use of the non-equilibrium formula, and this person endeavors to keep up with the latest developments in this field.

It is to be hoped that, before too long, the literature will have become more complete and stand-

ardized concerning the non-equilibrium theory and its application, so that the ordinary engineer or geologist, without help, can make and interpret pumping tests with reasonable assurance of not going off at a tangent.

In the meantime, the simple recognition of the time element in the ground-water problems and of the fact that there is a method for taking this element into consideration will go a long way toward making ground-water problems and their solution more understandable. The non-equilibrium concept, applied to an adequate set of data, with common sense and logical reasoning, can do a great deal more than has been done in the past toward getting good answers to ground-water problems and making sure that new developments are made on a sound and efficient basis.

That is the close of the prepared part. However, I have been asked to mention a little about the practical application of the non-equilibrium theory. The non-equilibrium theory is very practical, and we have applied it throughout the country. We have found, where we can apply it, where conditions are reasonable, that its applications have been justified. We can predict drawdowns within a very few per cent over a period of many years.

In one case at Lufkin, Texas, about 1939 there was a paper mill that put in wells and pumped about five million gallons daily. The water level plotted against time made a straight line declining. After about a year, the mill proprietors went to the U. S. Geological Survey and asked what to do, for they wanted to double the pumpage. The Survey told them it appeared that they were overpumping, but that the Survey would be glad to make pumping tests to make a better determination. The pumping tests were made. Also, we got all the records of pumpage and water levels, and we found that, although approximately five million gallons were being taken daily, the pumping rate had been stepped up just enough to affect the water levels so that they were changed from the typical recession curves to straight lines, when plotted on rectangular coordinates. The formation coefficients were computed. The available data indicated that there was a recharge area about 20 miles north. With this information, future recessions were computed for the proposed 10 MGD pumpage. New pump bowls and additional pump columns were put in the existing wells, and the pumping was doubled without any new wells. The predictions of water levels in the pumped wells have been checked within five per cent; and we checked it 20 miles away and found the difference between computed and actual drawdowns to be less than 15 per cent.

That is just one case. The paper mill people kept accurate records and they know exactly where they stand. They know how and where to get the water by putting wells here and here. They can decide from water level data how much to pump and what future levels will be.

There are other examples. At Camp Hood, Texas, we made pumping tests and predictions, and they checked the actual drawdowns after a year within three per cent.

There are some places where computations cannot be made so accurately. The concept is right, but the geologic conditions are too complicated. Trying to apply simplified assumptions to a radically different set of geologic conditions is not a fair test. The accuracy of the predictions of drawdown will depend on the adequacy of the mathematical assumptions as compared to actual conditions.

DISCUSSION

JOHN H. BLISS.*—Mr. Guyton's paper presents an excellent picture of the general situation regarding the availability and use of ground water in the United States and emphasizes the need for long-range programs for the collection of basic ground-water data.

As he states, excepting for local problem areas, there is no over-all shortage of ground water in the country. In general, the average annual recharge of such basins is ample for withdrawal requirements. In some areas, however, he points out that recharge of ground water is so limited that reasonable use of the resource must contemplate actual mining of the available supply. This is true in many areas of the Southwest. In New Mexico, for example, recharge of many of our ground-water areas is limited by inadequate rainfall, by surface geological formations which retard downward percolation of water or by a combination of circumstances. The limiting of pumping withdrawals to the safe annual recharge in such basins would probably place unreasonable restrictions upon their legitimate development. Ground-water mining in such basins can and should be permitted, provided that reasonable safeguards are observed and provided that the water users fully understand that the supply is not inexhaustible but will be largely exhausted in a relatively limited number of years and plan their economy accordingly.

Mr. Guyton tells of a problem area he studied last spring near Deming, New Mexico, where the recharge was very limited in amount and where the available ground-water supply was being rapidly exhausted. His study showed that the economic supply under present draft would be largely gone in a ten- to fifteen-year period—and the local people were frankly informed of this condition. Incidentally, I had the pleasure of working with Mr. Guyton on this case. His findings in the matter were of great assistance to the State in the solution of the particular question at issue.

New Mexico has a ground-water law which permits the State water official to administer and control development in any ground-water basin, stream or reservoir, the boundaries of which are reasonably ascertainable. It is patterned after the surface-water code but necessarily departs from it in many details. The sections setting forth the powers and duties of the administrative official are general and permit that officer rather wide latitude in making specific rules and regulations for administration of the law.

In his paper, the author points out that legal control of ground-water resources cannot be expected to do the whole job. He also emphasizes that too strict control may well stifle legitimate development, whereas too weak control is useless. The writer feels that the New Mexico law (though not perfect by any means) will permit effective development of the several ground-water areas of the state while giving reasonable protection to the prior water-right users in each basin who spent their time, effort and money in proving and developing its ground-water potentialities. The State has in every case encouraged prospecting and development of new ground-water areas. Although the State Engineer has the authority to define and administer all ground-water bodies of the state, he has not in recent years assumed jurisdiction over any basin where investigation or the stage of the water supply development has not indicated that control was required.

Unlike the users of surface waters, it is possible for later comers in a ground-water basin to take water from a fully or overappropriated basin without immediately affecting the long-established water-right users. This presents a considerable problem in ground-water administration because, though it might be legally proper to do so, it would be very difficult as a practical matter to force junior appropriators to cease their use of water after they had made considerable initial investments in their water projects, should subsequent studies and information show that such junior appropriators were jeopardizing the uses of prior water users.

In most of the nine basins now administered by the State Engineer, use of water beyond the dependable recharge of the basin has been permitted. The State Engineer has had to exercise considerable judgment in determining the extent of development which may be permitted in each basin and still provide prior appropriators with reasonable protection required by law. One criterion which has been applied (and which is based upon the contractual life of federal reclamation projects) is that early water users in a basin should, as nearly as possible, be assured of a usable water supply of at least 40 years duration. Whether this is a satisfactory criterion or not, time alone will tell. It is not necessarily a fixed one and can be changed, if necessary, if changed circumstances dictate that need.

Another problem of ground-water administration which has been hinted at in Mr. Guyton's paper is that of the interrelationship of ground and surface water supplies. In most states with ground-water laws, the two are considered as separate water supplies for administrative purposes. And yet, in many many areas, we know that there is a direct relationship between surface flows and the ground

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waters within the stream valley.

The author stresses the need for adequate long-range basic data-collecting programs and points out that lack of adequate basic information must inevitably result in fumbling, bad guesses, and improper development of available supplies. Basic ground-water data is somewhat analogous to stream-gaging records of surface-water supplies. In both cases, adequate long-time records are essential to an over-all understanding of the problems.

The latter part of Mr. Guyton's paper is devoted largely to the discussion of the Theis non-equilibrium theory. We in New Mexico feel that we have been particularly fortunate since Dr. Theis has been in charge for the U. S. Geological Survey of ground-water studies in the state since 1931, and was working in the state at the time he developed his non-equilibrium formula. It has been through his advice and counsel as well as that of other Survey officials such as A. G. Fiedler and S. S. Nye, that New Mexico was enabled to make the progress she has in the technical aspects of her ground-water administration. The Theis formula for determining the effect of pumpage from wells in the several basins of the state has been invaluable in analyzing the effect of existing programs and has served as an excellent yardstick in forecasting the results of ultimate basin developments.

In closing, I want to echo Mr. Guyton's hope that the latest literature on techniques of analysis of ground-water data, the application of the non-equilibrium formula, data on pumping test problems and other information may be correlated and standardized in the not too distant future so that the ordinary engineer or geologist may be able to apply them to ground-water areas with reasonable assurance of reaching the right answers. The need for such a manual or book is evident.

H. T. CRITCHLOW.*—Mr. Guyton has prepared an interesting and comprehensive paper on the analysis and use of ground-water data. From his service with the U. S. Geology Survey and as a consultant on ground-water problems, he has given us the benefit of wide experience in this field. He has dealt intelligently with reports and rumors regarding the seriousness of the ground-water situation throughout the country. He recognizes that the problem is serious in certain localities where overpumpage has caused depletion in the ground-water level beyond the dependable yield of the rate of recharge and where the contamination of the waters by salt-water intrusion has occurred.

In determining the total amount of ground water being used throughout the United States, he states that this use has tripled during the past 15 years from 10 billion gallons per day in 1935 to 30 billion gallons per day at the present time. I would question this rate of growth from my experience with ground-water problems in the East. For example, in New Jersey the consumption from wells is estimated to be around 400 million gallons per day, of which about 175 MGD is by public water supply systems. The records are not complete. It is generally true that the diversion of water from private wells is not accurately measured in most instances; therefore, many of the data are based upon estimates. This emphasizes the need for conducting comprehensive inventories of the private well owners and of urging them to keep accurate records of their water consumption. However, there is no doubt that use of ground water has increased rapidly in recent years. This has resulted from many factors. Radical improvements in the construction of the development of methods of treatment of ground water to soften it and remove iron and other objectionable materials which may be present, have done their part to increase the quantity and improve the quality. The demand for additional supplies of water in war and other emergencies has added considerable impetus. The result has been a situation which threatens the depletion of these valuable ground-water resources and the possible contamination and loss by salt-water intrusion and other sources of pollution.

Dr. Guyton states that legal control of the use of ground water is not the whole answer to the problem of conserving the supplies. He emphasizes that knowledge of the source and behavior of ground water is most important to the proper solution. Fortunately, much work has been done along these lines in certain areas and the results obtained should encourage similar studies in regions where ground-water problems are becoming serious. The U. S. Geological Survey has been the leader in this field. It began investigating ground-water resources of the country soon after its establishment in 1879. Its first report on the subject was published in 1885. In 1903 the Division of Hydrology was formed, which was later renamed the Division of Ground Water. Funds for this work were meager for many years, but in 1929, as a result of demands by the states for aid, Congress increased appropriations for ground-water investigation for use primarily in cooperation with state, county and municipal governments on a dollar-for-dollar basis. As a result, the Division of Ground Water has cooperated for over 20 years with many of the states on a coordinated program which has favored the development of an effective technique in this highly specialized scientific field with its practical applications. Another result has been the selection and intensive training of capable geologists, engineers and physi-

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cists in the field.

As a result of questionnaires sent to all state water agencies by the leaders of this conference, replies to date from 26 states show that 23 of them cooperate with the U. S. G. S. on ground-water studies. A few brief statements about the experience in New Jersey, I think, will emphasize the importance of basic data in endeavoring to deal with ground-water problems.

Investigational work in New Jersey began in 1923 and has continued without interruption to date. From the beginning, this was carried on in cooperation with the U. S. G. S., under the sponsorship of the State Water Supply Commission and its successors in name due to several reorganizations of state agencies—since 1945 it has been under the Division of Water Policy and Supply of the Department of Conservation and Economic Development. During this period of 28 years, field investigations have been conducted in ten selected areas within the state, for the main purpose of determining as far as possible the dependable yield of the water-bearing formations and the source of salt-water contamination. The total funds expended to June 30, 1951, was about $490,000 by the State and Federal governments, and for the current fiscal year $49,000 have been appropriated. The field investigations include the operation of 265 water-level stations and 153 water-sampling stations for chloride analyses. The water-level stations comprise 84 continuous recorders, 40 periodic (monthly), and 164 occasional (quarterly) measurements of water levels in observation wells in the areas under study. Many of these wells have been drilled especially for observation in selected locations. Pumping tests are conducted to observe yield, drawdown, cone of depression, interference with neighboring wells, and other pertinent data. Logs of geologic formations, including samples of materials, are obtained and correlated. Samples of water are collected for complete chemical analyses, especially from new wells, and thousands of samples have been taken from wells along the coast for chloride analyses to obtain advance notice on the approach of salt water.

Another very practical use of the results of these investigations is to assist the Water Policy and Supply Council in the administration of the laws on the control of ground waters. This control has been continued since 1910, when a law originally passed in 1907 regulating the diversion of surface waters for public use was amended to extend the jurisdiction over diversion from wells, subsurface and percolating waters. Under this legislation the State agency allocates new or additional sources of water supply, upon application and after public hearing. The applicant is required to show (1) whether the plans proposed are justified by public necessity; (2) whether they provide for the proper and safe construction of all works connected therewith; (3) whether they provide for the protection of the supply and the watershed from contamination or provide for the proper treatment of such supply; (4) whether the reduction of the dry season flow of any stream will occur to an extent likely to produce unsanitary conditions or otherwise injure public and private interests; (5) whether the plans are just and equitable to the other municipalities and civil divisions of the State affected thereby and to the inhabitants thereof, particular consideration being given to their present and future necessities for sources of water supply.

For forty years of operation of the statutes on control of waters for public and potable use in New Jersey, there was no State control over private wells for industrial, commercial, or other private uses. Public supplies were thus subject to unrestricted interference by private wells that might be constructed and operated within their sphere of influence. For ten or more years relief was sought through legislation. Finally, in 1947, a law was enacted giving the State agency authority to regulate the diversion of subsurface and percolating waters of the state for domestic, industrial and other uses. The law provides:

1. The Department shall delineate from time to time such areas of the state where diversion of subsurface and percolating waters exceeds or threatens to exceed, or otherwise threatens or impairs, the natural replenishment of such waters.

2. In areas so delineated no such waters shall hereafter be diverted in excess of 100,000 gpd. for any purpose without obtaining a permit. Such permit may be refused or, if granted, may include such stipulations as may be necessary to conserve such waters of the state and prevent their exhaustion.

3. Any refusal to grant a permit shall be subject to review by the Supreme Court, both as to question of law and fact.

4. Any person, corporation or agency diverting in excess of 100,000 gallons per day from such sources shall have the privilege of continuing to take from the same source the quantity of water which is the rated capacity of the equipment at that time used for such water diversion.

Under this law, passed in 1947, four protected areas have been established where investigation of ground-water conditions and experience of users has required the establishment of control as provided by the law.

The demands on ground-water supply for industrial, municipal, agricultural and domestic purposes are growing out of all proportion to the increase in population. That is particularly true in our own state. As previously stated, the enlarged demand has been due principally to methods of well construction, deep-well equipment, which has been designed and built, efficiency of pumps and motors

and lower power rates. Rapid growth in the use of air-conditioning equipment in many cities is increasing the draft on wells and overloading drainage systems. This situation is bringing about the passage of laws and regulations to require the conservation of ground water by recirculating it through cooling devices or returning it to the ground through recharge wells.

The need for more scientific knowledge about the occurrence and movement of ground water is vital to a satisfactory solution of dependable yield, interference, contamination, and other problems. The program inaugurated by the U. S. G. S. in cooperation with the states and other local agencies has been of great help in the economic development of ground-water resources. Whether the states carry on investigational work in cooperation with the federal agency or through their own efforts is a matter for each to decide for itself. The important thing is to do it in a scientific and continuing manner. Research is the foundation of technical progress and pays large dividends. As increased demands are made upon ground-water resources, reasonable legal control by a state agency based on scientific knowledge and experience would seem to be the best solution of this important problem.

H. A. SPAFFORD.*—I want to commend Mr. Guyton on this very splendid paper and his straightforward and practical presentation, describing some of the tools we can use in arriving at the solution of some of our ground-water development problems. The Illinois State Water Survey is utilizing these tools to help a lot of us arrive at some of the answers we need to have. The Survey has been of great assistance, in applying these basic tools and interpreting data, to many of us, in arriving at some equitable answers in the development of public ground-water supplies in this state.

There are approximately 1100 incorporated municipalities in Illinois and at the present time about 750 have public water supplies. Approximately two-thirds of these have been developed utilizing ground-water sources. There are still approximately 350 incorporated communities that do not have public water supplies. These range in population from a few villages having under 100 people to a maximum of one city having slightly over 1000 people without public water supply facilities. The majority of these potential communities yet remaining without public water supplies are located in what some of us choose to term "lean" ground-water areas, or where waters may be so highly mineralized as to be unusable and economically untreatable by our present known methods. In spite of this, we are still developing new public water supplies within this state. We are averaging about 10 every year and that average has been fairly consistent over the last 20 years. In the last three years 39 new public water supplies were developed; about one half were in lean ground-water areas. The State Water Survey has assisted, and my department has been very grateful for this help, in the development of these new public supplies, particularly in these lean areas.

Often some of these supplies are developed from wells, that some of you would term "piddling," at production rates of 15 or 20 gallons per minute, or less. One supply developed about one year ago for 450 people utilizes five wells, each producing 6 gallons per minute. Another town will place in operation, in a few months, wells producing at rates of 5 or 6 gallons per minute. These particular wells obtain water from a relatively shallow sandstone. The pump tail pipes are to be set no deeper than the top of the water-bearing formation so they cannot be pumped below the top of the producing formation. It is interesting to note that on these wells the driller reported a yield of 15 gallons per minute for periods of 24 hours, but later when the State Water Survey made carefully controlled tests on them, they evaluated the safe long-time production rate at 6 gallons per minute. One of the things I want to stress is the fact that in these lean ground-water areas, we have to accept the situation that they are lean. We cannot attack the problem with the idea we are going to have to develop wells capable of delivering 100 or 200 or 1,000 gallons per minute. Development of public supplies is most important to those small communities who have limited economic means to provide them. Therefore it is essential that we cut corners as far as necessary within reasonable limits, to make it possible for them to provide supplies so important to their well-being and growth.

We try to be both practical and reasonably conservative, in our design standards, for these small communities. Usually we use a minimum-per-capita design basis of about 50 gallons per person per day. I realize that most of you have been thinking of 100 gallons per capita per day or more. Our studies show that these small communities without industries, rarely use more than 50 gallons per capita per day.

Some supplies have been developed from shallow sand and gravel deposits, subject to local recharge. In these instances we want these pockets proved by test drilling and pumping so we are sure there will be a minimum of six months' supply available to tide over a drought period, which is about the maximum length of drought period experienced in Illinois. It may be shorter or longer in other areas.

There is another tool, I do not believe Mr. Guyton mentioned, that has been very valuable in

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Illinois. That is the use of electrical earth resistivity surveys. In local sands and gravels that may be water-bearing, these surveys have made it possible to eliminate much unnecessary blind drilling of test holes. Some extremely valuable ground-water resources have been located in this manner and the number of test holes in these areas has been greatly reduced, by the use of earth resistivity surveys.

E. W. BENNISON.*—Dr. Guyton's paper was very fine; it brought us up-to-date on the theory. I have been connected with the practical aspects, using what theory was available. The question that came to my mind is the limited application of that to the practical problems of the day. I have discussed these methods with men practicing in the field. There is a problem of getting these methods applied. We see ground-water developments being made every day which are generally economic propositions. We see the engineers of the country faced with developing more water quickly and certainly little use made of such formulas like those Dr. Guyton gave. I would like to know how to bring the theory and practice together.

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W. F. GUYTON.—Certainly no one knows any more about the practical developments than Mr. Bennison. Most of you have seen his book and literature.

These formulas are practical, providing you can meet the assumptions on which you make your computations. They do not help you build a well. That is another part of the problem. If we can work out the geology to put into the formulas, the theory will predict the amount of well interference over a period of years.

We have had more success in the widespread formations in the South, the formations that occur over a good many miles under conditions that are more or less uniform. In south Texas, over a distance of 20 miles, we consider the predictions accurate enough. When you come here to another type of geology, it may be harder to use this method.

All the work in this field is still in a state of experiment, but every year we get a little closer. It has become evident to me, however, that a knowledge and appreciation of the non-equilibrium theory is of the utmost value in understanding the data we collect in everyday developments, so that we can make these developments more economical and dependable.

ANALYSIS AND USE OF SURFACE WATER DATA

BY RICHARD HAZEN*

WITH DISCUSSIONS BY E. F. BRATER, W. D. MITCHELL, AND C. V. YOUNGQUIST

NEED FOR SURFACE WATER DATA AND ANALYSIS

Surface waters play an increasingly important role in America. Rivers are dammed for water supply and irrigation systems and for the generation of power. Navigation channels are maintained over many parts of the country. Flood control works are built, but the menace of devastating floods continues in many areas. Stream pollution is alleviated by supplementary low river flows from reservoirs. In the proposed Incodel project for the Delaware River, more than one half of the storage is intended to improve the water quality downstream and to prevent the encroachment of salt water from Delaware Bay.

Stream flow is never constant, and to a certain extent it is unpredictable. The mean runoff of a stream is its most important characteristic. It represents the maximum quantity of water that theoretically can be developed. In practice, works can never be built to utilize fully the mean runoff. Evaporation and transpiration losses are unavoidable, and large flood flows are not often fully utilized. Most surface-water developments are designed to equalize the flows of a river—to reduce the peaks and to increase the dry weather flows. Stream-flow records and analyses are needed to plan properly the construction and operation of such works.

The rapid growth in surface water works has brought about a large increase in the number of persons engaged in hydrological analysis, and in the volume of data available. Earlier investigators had only limited data to work with and were obliged to develop means for extending available information. Many estimates were approximate at best. The inadequacy of some has been proved time and again by the failure of dams during high floods or the lack of water during extended dry seasons. (It should be noted, however, that many shortages have been due not to improper estimates but rather to a failure to build additional works in time to meet rising water demands.)

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EFFECT OF CHANGED CONDITIONS ON RUNOFF RECORDS AND PROBABLE ACCURACY

Most stream-flow analyses are made in order to estimate future flows. Past flows are not important except as a guide to what may happen in the future. Normally, we assume that future flows will follow the pattern of the past. For most practical problems this is a fair assumption, although long-term changes in the weather, including temperature and rainfall, and in surface runoff, are recognized.

Studies by the U. S. Geological Survey indicate a gradual decline in the rainfall and runoff over much of the Missouri River Basin during the past seventy years. The long-term rainfall trend at four selected stations is shown on Figure 30. The tendency may be reversed within a short time, and the length and frequency of wet and dry spells cannot be predicted accurately on the basis of cycles. As our records extend over much longer periods, the laws governing these cycles may become apparent, and it is possible that adequate explanations will be developed even without the benefit of such data.

The design of engineering works is rarely affected by changes in runoff to be expected several hundred years hence. Moderate allowances to take care of changes that may develop within shorter periods normally will suffice. Where data are available for only a short period, say twenty or thirty years, it is important to know whether or not the record reflects a relatively wet or relatively dry period, and if extremely high and low flows occurred during the period of record. This usually can be established qualitatively, if not quantitatively, by the study of the longest rainfall records in the vicinity. Generally, stream-flow and rainfall data are inadequate for accurate estimates of runoff on the basis of rainfall. In the eastern and midwestern United States the years 1930-32 and 1939-42 were unusually dry. From long-term rainfall records the twelve-year period 1930-41 is considered the driest to be expected in the upper Missouri River valley over as much as three hundred years. Regardless of the exact frequency, it is safe to assume that these dry periods do represent extreme conditions; thus, even relatively short records may be

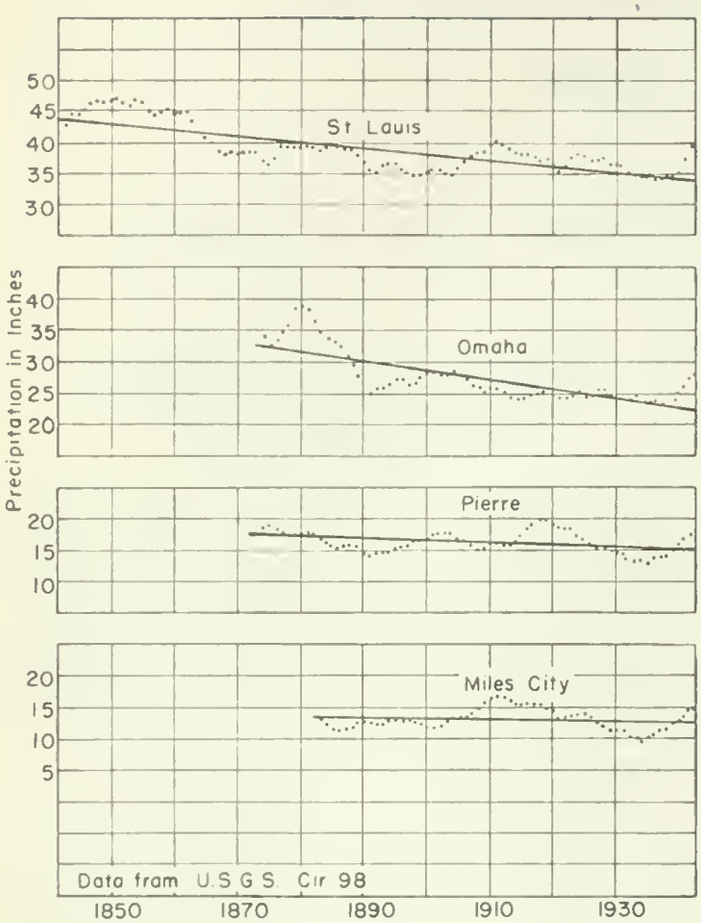


FIG. 30.—PLOT OF 10-YEAR CENTERED MOVING AVERAGE ANNUAL PRECIPITATION WITH COMPUTED TREND LINES FOR SELECTED WEATHER STATIONS.

suitable in many areas for estimating low flows.

Man-made changes in runoff cannot always be neglected. They come quickly and their effect upon stream flow may be pronounced. One of the difficulties in stream-flow studies is the absence of long-term records not affected by the construction of reservoir or diversion works. Where the quantities of water stored or diverted are carefully measured, the flow records can be adjusted. Accurate adjustment is often impossible. For example, errors in estimating the evaporation from a large reservoir surface during a dry season may be considerably greater than the stream flow. Upstream storage or diversion is particularly important in regard to low-flow analyses and storage studies. They are usually less significant in flood-flow studies because only large storage projects have appreciable effect upon the peak flows. It has been estimated for eastern rivers that storage capacity sufficient to regulate completely the flow from 3 per cent of the drainage area is needed to effect each 2 per cent reduction in flood quantities.

Earlier stream flow records are occasionally revised to show corrections in rating tables and drainage area measurements. The records published by the Geological Survey include summaries of average and extreme flows and comments upon upstream regulation and the probable accuracy of the record. In using these publications, it is well to start with the latest record and to work backward so that corrections can be made promptly without repeating considerable work. For detailed studies of low flows it is important that the upstream conditions be checked carefully. A certain mill in Pennsylvania takes water from a spring with a natural outlet to a nearby river just below the U. S. Geological Survey gaging station. When the mill is operating, the spring water goes through the mill and into the river above the gaging station, and the spring discharge is included in the stream-flow measurements. When the mill is not operating, the spring discharge is not measured. At periods of low river flow the spring discharge is an appreciable part of the total, and it is thus necessary to determine on which days the mill was operating, so as to properly adjust the records.

The U. S. Geological Survey rates as "excellent" records in which the error in the daily flow is believed to be less than 5 per cent, "good" where the error is less than 10 per cent, "fair," less than 15 per cent, and "poor" where the error is probably

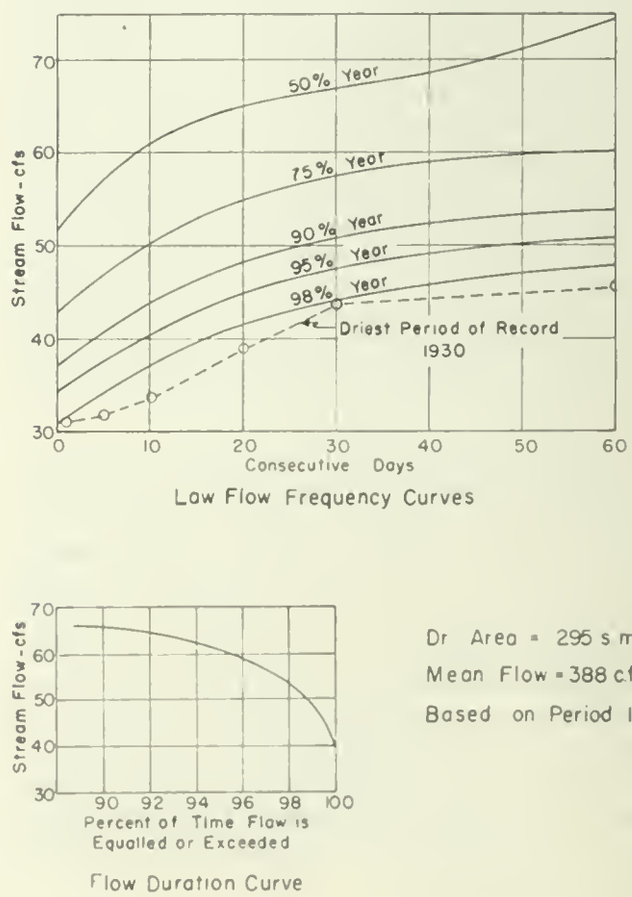


FIG. 31. — FRANKSTOWN BRANCH, JUNIATA RIVER AT WILLIAMSBURG, PENNSYLVANIA.

more than 15 per cent. In general, the records of monthly and yearly mean discharge are more accurate than the daily record, since the errors in the daily measurements tend to offset one another. The probable accuracy of the stream-flow data under analysis should be evaluated before any tedious detailed studies are undertaken. A particular warning is made against the use of the difference between daily measurements on the same stream in order to estimate the inflow between the two gages. Obviously, if the readings at one station are consistently high and at the other station consistently low, the calculated difference may be far from the truth. Flood flows are hard to estimate accurately at many stations, and the records may be in error by as much as 25 or 50 per cent.

LOW FLOW STUDIES

In most surface water studies we are interested in extreme flows—either dry weather flows that need to be augmented or flood flows that need to be curtailed or released without damage. The flow characteristics of a river are often indicated by means of a flow duration curve, usually prepared from the daily flows over the whole period of record. If the investigation is limited to low flows, it is necessary to plot only the lower end of the curve and the volume of work to be done is greatly reduced. Figure 31 includes the lower end of a duration curve showing the flows exceeded 90 per cent of the time, or more, in the Frankstown Branch of the Juniata River at Williamsburg, Pennsylvania.

The duration curve shows the runoff characteristics of a stream and how often flows of various magnitudes can be expected. Even within areas of substantially the same rainfall the runoff pattern may vary widely, depending to a large extent upon the topography and natural ground storage. Thus, for example, the low flows of the Mad River in Ohio are relatively much greater than the low flows of the Hocking River. In some parts of the country there is little or no natural ground storage, and the rivers dry up entirely for weeks at a time. The Division of Water in the State of Ohio, with the cooperation of the U. S. Geological Survey, has prepared duration curves for all of the longer records in the state. The flow, exceeded 90 per cent of the time, has been designated as the "index of flow." For the Hocking River at Athens the index is 0.088 c. f. s. per square mile; for the Mad River at Springfield the index is 0.340 c. f. s. per square mile. The "index of flow" shows quickly whether a stream has relatively large or small ground storage.

A duration curve of daily flows does not indicate how the days of low flow were arranged, and whether or not they were interrupted by freshets. Thirty consecutive days during which stream flows averaged 50 c. f. s. may be of greater practical significance than thirty days with daily flows less than 50 c. f. s. distributed over several months. Extreme low flow conditions may be shown by plotting the driest week, the driest thirty days, etc. The preparation of such graphs is laborious for streams with long records because there may be several periods of almost equal dryness, and the driest cannot be determined without studying each period in detail. Furthermore, the driest consecutive data do not tell how often periods of the same or nearly the same intensity occur, frequently an important consideration in water supply, hydroelectric power, and stream pollution investigations.

A combination of the duration curve and the driest consecutive plotting can be obtained by subjecting the driest consecutive data for each year to a probability or statistical analysis. The results of such an analysis for the Frankstown Branch are shown on Figure 31. The lowest curve represents the driest period of record, which corresponds very nearly to the 98 per cent condition. The record is not long and the probability plottings are fairly irregular. The curves shown on Figure 31 have been smoothed out to show the general trend. The analysis described above still does not show the effect of two or more extremely dry periods within the same year. However, except in the event of unusual upstream regulation or flash floods, the lowest flows usually occur in the same general period, toward the end of a long dry spell.

STORAGE RESERVOIRS

Storage requirements are usually determined from mass diagrams or by arithmetic using the runoff data directly. Mass diagrams of daily or monthly flows are used. The daily plotting results in slightly greater storage requirements, but the difference is not large for relatively high developments involving large storage with a carry-over of a year or more. Sometimes diagrams of the cumulative monthly deviations from the mean flow are used. This plotting is particularly helpful in showing long-term tendencies. A mass diagram of stream flows cannot be used conveniently for estimating storage needed to maintain flows low in relation to the mean flow. Either the scale is too small to take off the storage quantities accurately, or if the scale is made large enough, the diagram becomes too bulky. In this respect a diagram of the cumulative deviations from the mean is preferable, and frequently a hydrograph of flows will give the answer as quickly as any other method. In general, plotting the data is desirable because if calculations are made directly from the runoff tables, errors are more likely to go unnoticed.

As the stream flow records become longer, the tendency is to estimate more and more the storage requirements upon the driest period of record. If

data are not available for the stream in question, records for neighboring streams are used. This procedure is satisfactory as long as the records include at least one period of extremely low runoff, but reliance upon a single instance leaves much to be desired in any event. A statistical or probability analysis of the storage required each year during the period of record makes it possible to estimate the probable frequency of the droughts recorded and the degree of storage that will be needed. This type of analysis is helpful also in indicating whether or not the extreme conditions recorded are better or worse than to be expected normally over the number of years included in the record. In the eastern United States the storage required in the driest years is usually not more than two or three times the storage required in a year of average runoff, and the variations in storage follow the normal law of error quite closely. In this respect the statistical method is generally more satisfactory for storage analyses than for flood flow estimates.

It should be noted that estimates of storage requirements can be a little more flexible than flood flow estimates. Storage reservoirs are built to be used over a long period in the future, normally with the expectation that water requirements will grow. If the storage estimates prove unduly optimistic, it is usually possible to provide additional capacity before a water shortage develops. Furthermore, water use can be curtailed in a pinch, or if the reservoir is used for pollution control, a lowering of standards for a short period will not ordinarily have serious consequences. On the other hand, the capacity of a spillway must include a large margin for safety, since failure of the spillway might lead to overtopping and destruction of the dam.

In his early paper on the application of probability methods to storage problems and in the tables included in the Merriman-Wiggin American Civil Engineers' Handbook, Hazen suggested that the 95 per cent year be used in estimating storage requirements for impounded water supplies. The 95 per cent year is a relatively dry year, representing conditions to be expected on the average only five years in one hundred or one year in twenty. The extremely dry periods in long records of eastern streams represent approximately the 98 per cent year. Cheap money rates, the rapid increase in per capita water consumption (about 35 per cent since 1925 in many American cities), and the uncertainty as to future costs, have encouraged the construction of reservoirs considerably larger than indicated by the 95 or 98 per cent condition. Frequently much larger reservoirs have been built not for the additional storage but rather to obtain gravity flow. The basic concept of limiting storage capacity to that required in relatively dry years, but not necessarily the very driest years, is sound. It is directly comparable to the design of storm drainage facilities that we know will be flooded from time to time. The economic reasoning behind this procedure has been obscured in recent years by continuously rising costs. Under present conditions of high costs, it is natural to lament the failure to build much larger works twenty or thirty years ago at relatively low prices.

A revision of the American Civil Engineers' Handbook is underway, and we have been reviewing the 1930 storage tables in the light of later and more extended data. These tables set forth so-called "normal storage" requirements based upon statistical analyses of a relatively few stream-flow records available at the time. The storage requirements can be estimated from the tables by determining the draft in relation to the mean flow, and the coefficient of variation of the annual flows. Corrections for varying degrees of ground storage must be applied. The tables work well for eastern and Appalachian Mountain streams. This is to be expected since most of the longest records used in preparing the tables came from the East. Mr. W. W. Horner has pointed out that in the Middle West the correlation is not good. The "normal storage" curves and the storage estimated from the driest period of record for several midwestern

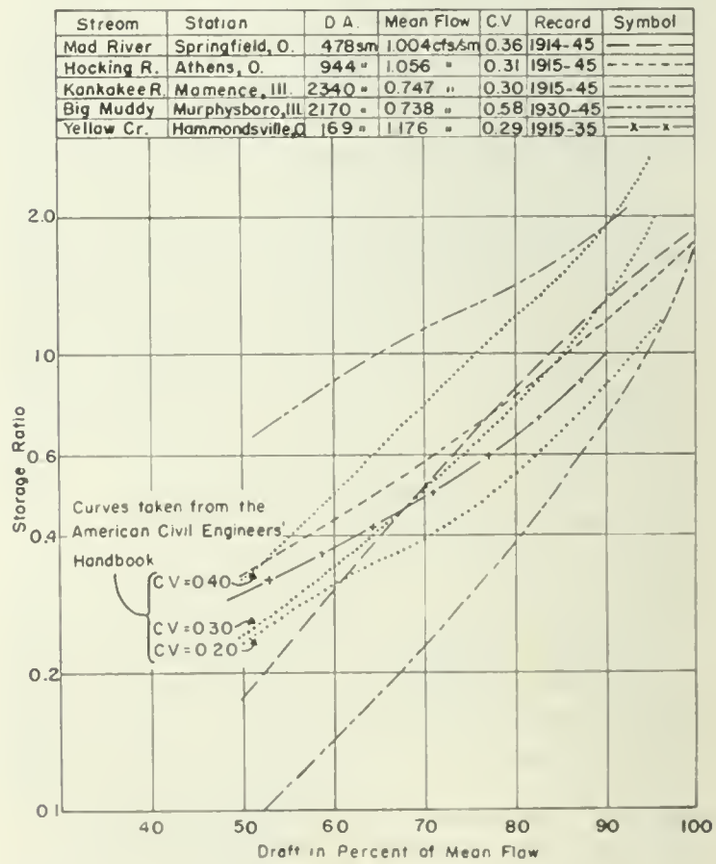


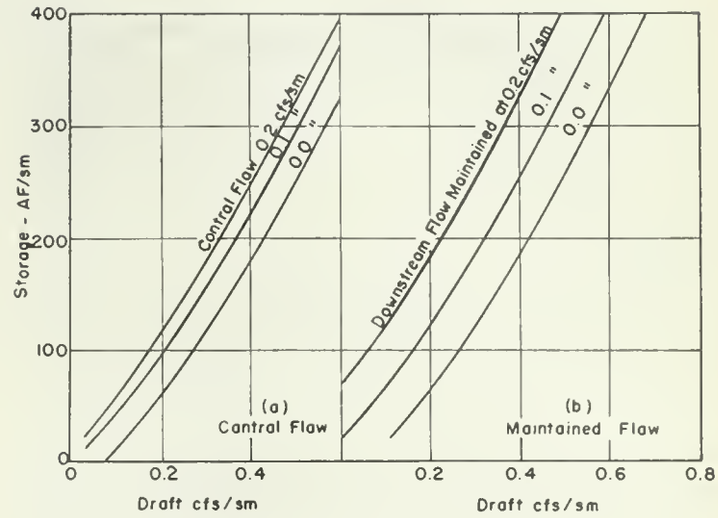
FIG. 32.—STORAGE CURVES OF FIVE MIDWESTERN STREAMS COMPARED WITH CURVES FROM THE AMERICAN CIVIL ENGINEERS' HANDBOOK.

streams are plotted in Figure 32. Since the driest period of record is not necessarily the 95 per cent year, direct comparison should not be made, but this difference is not large for the records used. It will be noted that except for the Kankakee River and the Mad River, both of which have considerable natural ground storage, the actual storage curves are considerably flatter than the "normal storage" curves. At relatively high developments the agreement is fair, but at lower developments the normal curves do not give enough storage. Dry periods of several months duration with little or no flow in the streams are more common in the Middle West than in the East. A series of curves fitting midwestern conditions more closely can probably be developed. In the Rocky Mountains variations from year to year and from wet to dry periods of several years are large and a different relationship is required.

DOWNSTREAM WATER RIGHTS

Downstream water requirements play an increasingly important part in water supply developments. There are few places in the United States where all of the water from a stream can be stored or diverted without consideration of the lower water users. In the West where the appropriation doctrine has largely supplanted the riparian rights doctrine, the lower water rights are established by court decree, and the excess water available, if any, is pretty well established. In the East where in general the available supplies of water have exceeded the water demands, water rights quantitatively are not so well defined.

In a recent study of water resources for potential synthetic liquid fuel plants in twenty-six states and Alaska, it was necessary to consider a great many rivers and potential reservoir sites, and the effect upon downstream users. Special account was taken of important water uses where these were known and in the Rocky Mountain states where appropriations are recorded. Elsewhere a uniform procedure was followed. The stream flow per square mile of drainage area, normally exceeded 90 per cent of the time, was determined from the daily runoff records. This flow, which is the same as the flow index in the Ohio studies, was called the "control flow." It was then assumed that no water could be diverted from a stream whenever the natural flow was less than the control, or 90 per cent, flow. Where storage reservoirs were necessary it was anticipated that whenever the natural flow was less than the control flow, water would be released from the reservoir in quantities equal to the natural inflow. It was not assumed, however, that any part of the storage would be used to augment the low flows downstream. The effect of the control flow allowance on the capacity of the reservoir is illus-



Note Under (a) stream flows less than the control flow are allowed to continue unchanged. Under (b) reservoir capacity would be used as necessary to maintain the indicated downstream flow.

FIG. 33. —STORAGE CURVES FOR THE HOCKING RIVER AT ATHENS, OHIO (BASED ON DRIEST PERIOD OF RECORD).

trated in the left-hand portion of Figure 33, which shows the storage required per square mile of drainage area for the Hocking River during the driest period of record, 1930-32. The control flow in this case was 0.088 c.f.s. per square mile, or very nearly the 0.1 c.f.s. shown on the graph. For comparative purposes a control flow of 0.2 c.f.s. per square mile is shown also, although this value was not used in any of the synthetic liquid fuel studies.

On the right-hand side of Figure 33 is shown the storage required if part of the reservoir capacity were to be used to supplement the natural flow below the reservoir, as contrasted with the former case in which it was assumed that the low flow conditions would be the same as if no reservoir had been built.

The storage required for a draft of 0.4 c.f.s. per square mile under these two conditions is compared in Table I.

The procedure used in the synthetic liquid fuel studies is not suggested as one applicable to any particular situation. It was recognized that the uniform allowance would be too small in some cases and greater than necessary in others. For a preliminary survey, however, the procedure seemed reasonable and called attention to the necessity for taking into consideration downstream users.

Figure 34 shows the average, maximum, and minimum storage curves for twenty-eight streams studied in the synthetic liquid fuel survey in the Appalachian Mountain states and fifteen rivers in the Midwestern states. These curves were determined from mass diagrams of the driest period of record after deducting the control flow or 90 per

Table I

Control Flow c. f. s.	Storage AF/s. m.	Req'd Ratio	Maintained Flow - c. f. s.	Storage AF/s. m.	Req'd Ratio
0	180	1.00	0	180	1.00
.1	225	1.25	.1	255	1.42
.2	250	1.39	.2	330	1.83

cent flow described previously. The variations in the storage requirements are considerable. Some of these variations could be reduced or eliminated by converting all of the data to terms of mean flow. Others, of course, are caused by widely different drainage basin characteristics. In general, streams having large natural ground storage require smaller reservoirs to maintain a given draft per square mile of drainage area. However, in highly developed watersheds where a substantial proportion of the stream runoff is to be used, the ground storage is of less importance. This is because the critical conditions develop at the end of long dry periods, when the quantity of water available in ground storage is considerably reduced. A satisfactory relation between the 90 per cent flow, or some other low-range flow, and the storage required in an average year probably can be established. However, the correlation between such flows and the storage required during an extremely dry period is not good.

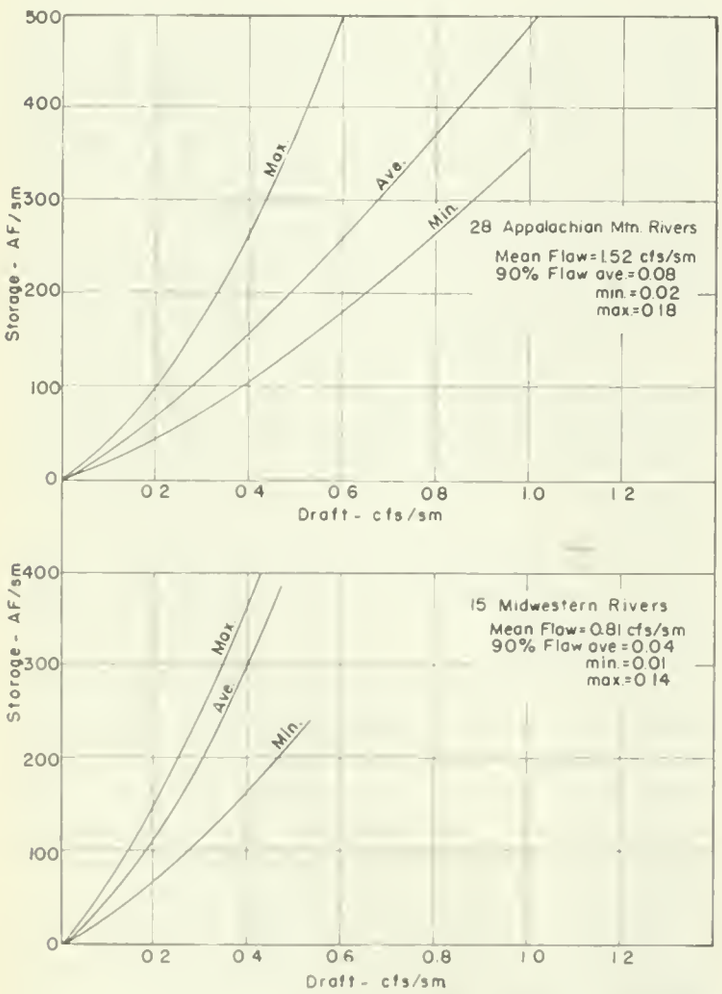


FIG. 34. — STORAGE CURVES BASED UPON DRIEST PERIOD OF RECORD AND RELEASE OF ALL FLOWS BELOW THE 90% FLOW.

In many areas it was expected that the water supply for a synthetic liquid fuel plant would be taken from a relatively large river, adequate for all demands except during a few weeks in the dry months. This type of development was considered for the Wabash River, the Allegheny River, the Monongahela River, and several others. For these rivers potential dam sites were considered on tributary streams. The limiting factor in the development was frequently the ability of the available drainage area behind the dam on the tributary to refill the reservoir promptly after a dry period. A series of curves were plotted for each drainage basin relating the yield, storage, and percentage of the total drainage area impounded with limiting values set to assure refilling of the reservoir. In these studies ten-day average flows were used in estimating the storage requirements. Monthly data used in analysis of relatively high developments were not adequate to show storage requirements for short periods of time. Daily data might better have been used, but several studies showed that the ten-day records were sufficiently close for our purposes, and by their use the work was reduced considerably.

DIVERSION WORKS

It is sometimes necessary to determine the capacity of works such as canals, tunnels, and pumping stations to divert a certain quantity of water from one river into a reservoir on another drainage basin. Normally it is not economical to divert all flows because diversion works equal in capacity to the maximum flood are impractical. At extreme

low flow periods the available water may be very much less than the diversion capacity. Therefore, the works must be built to take the water when it is available up to reasonable limits. The capacity to be provided and the quantity of water that can be diverted can be determined by analyzing the daily flow records. In general, the variations from year to year follow a pattern, which is similar for streams in the same general area. If storage is provided at the diversion works, the effect of minor variations in flow can be eliminated.

Where downstream water users must be considered the quantity of water that can be diverted is reduced. On Figure 35 is shown the diversion capacity required in the average year and in the 95 per cent year for diverting water from Wallkill River, a tributary of the Hudson River in New York State, based on the stream flow records from 1921 to 1935. These curves show not only the capacity required if there is no downstream discharge, but also how much additional capacity must be provided to take care of various downstream releases. It will be noted from these curves that increasing the diversion capacity above two or three times the mean flow does not increase appreciably the amount of water that can be diverted. In general, a diversion capacity three times the mean is about the economic limit, unless water is extremely valuable. The effect of releases downstream is very pronounced.

Figure 36 is taken from a diagram prepared many years ago by F. H. Hapgood of Hazen, Whipple and Fuller, in connection with the Manhan River in Massachusetts. These curves show the effect of providing storage capacity at the diversion works. The benefits of even a small amount of storage at the diversion works are apparent. It will be noted here also that a diversion capacity of about three times the mean flow will make it possible to obtain 80 per cent of the water and that further increases in capacity show a much lower return.

In estimating yields from these curves it is assumed that the diversion works would be operated

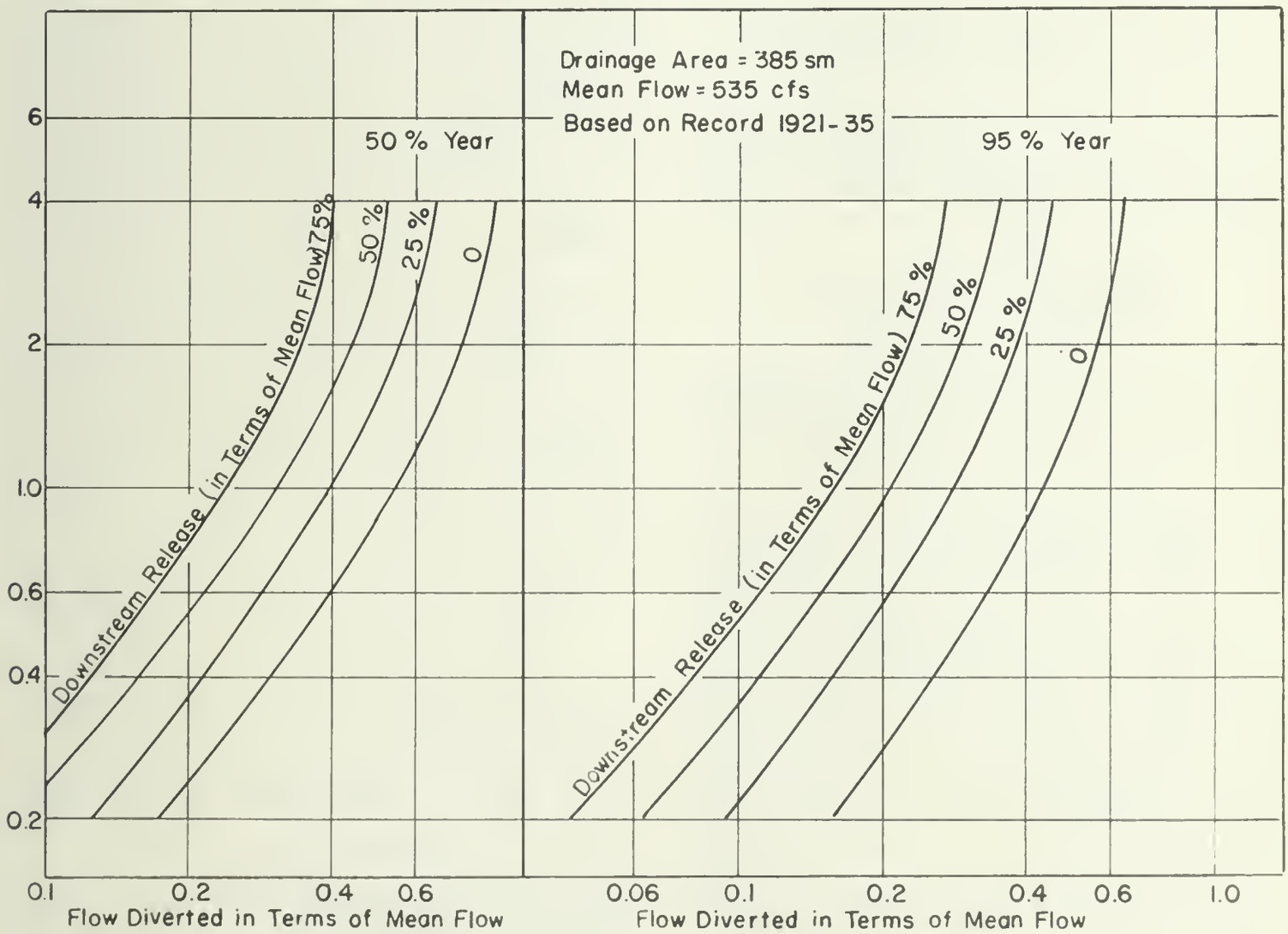
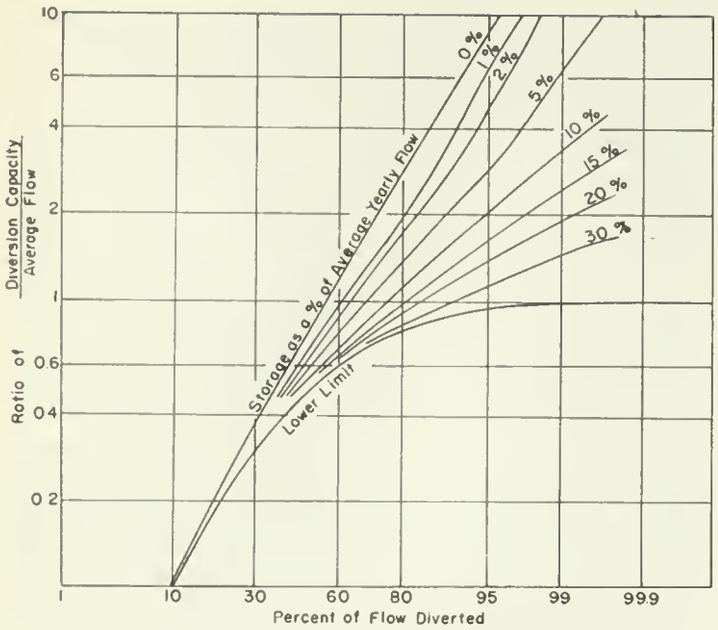


FIG. 35. —DIVERSION CURVES, WALLKILL RIVER AT PELLET'S ISLAND MOUNTAIN, NEW YORK (NO STORAGE).



Calculations based on daily flow; no allowance made for hourly fluctuations. To cover these, some extra storage should be provided. Calculations by F. H. Hopgood. Traced from Dwg. No. 19201 of Hazen, Whipple & Fuller

FIG. 36. —DIVERSION CURVES, MANHAN RIVER, MASSACHUSETTS, 1897-1915 INCLUSIVE.

to take all of the water available up to their capacity. In other words, gates on a gravity canal would be kept wide open, and pumps would be operated continuously as long as water was available. It is also assumed that the receiving reservoir could store the water.

EFFECT OF STREAM FLOW ON WATER QUALITY

Some mention should be made of the growing importance of water quality and the effect of stream flows upon water quality. The correlation between mineral concentrations in river water and the volume of flow is usually good. Frequently the low flows are contributed to a large extent by springs or from ground water, and during periods of low flow, the water is hard. High flows represent more rapid surface runoff, reducing the mineral concentrations, and frequently increasing the turbidity. The effect of river flow on hardness is shown for the Frankstown Branch of the Juniata River in Figure 37 and on the total dissolved solids in the Arkansas River on Figure 38. The biochemical-oxygen demand and dissolved oxygen, important in stream pollution studies, are affected by temperature and reaeration, and usually are not so closely related to stream flows.

The Water Quality Division of the U. S. Geological Survey and cooperating states have conducted an effective program in collecting river water analyses. Ten-day composite samples are taken for a period of a year or two at stations on the main streams, and spot samples are taken elsewhere at periods of high and low water. With these data a clear picture of water quality and its variations during the year can be obtained. Except where river quality is affected seriously by changes in industrial wastes, strip mining, and sewage disposal, the water quality in most large rivers does not change materially from year to year. Therefore, continuous analysis of most river waters over a long period of years is unnecessary.

FLOOD FLOWS

This paper has touched only briefly the subject of flood flows. In estimating floods it is usual to consider the maximum floods of record, not only for the particular stream in question but also for streams in the adjacent area. Allowances well beyond the worst flood of record are frequently made, depending upon the purpose of the estimate. Probability analyses of floods indicate the frequency or average recurrence interval to be expected for a flood of given magnitude. Most flow records are relatively short, and extrapolation of the data to

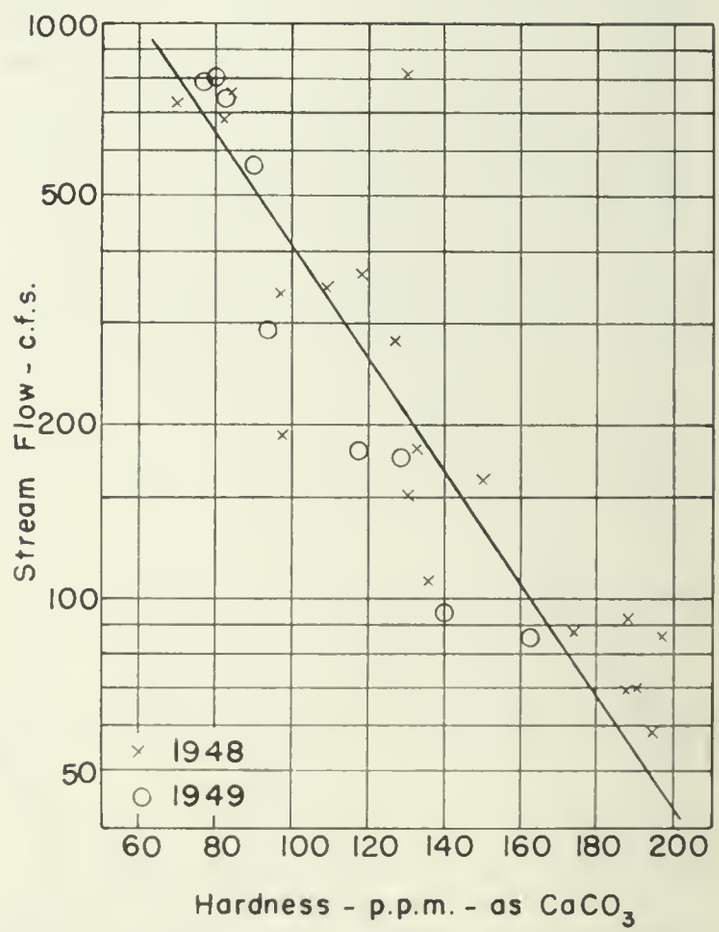


FIG. 37. — RELATION BETWEEN RIVER WATER HARDNESS AND STREAM FLOW, FRANKSTOWN BRANCH, JUNIATA RIVER AT WILLIAMSBURG, PENNSYLVANIA.

cover recurrence intervals of 500 or 1000 years is dangerous.

Extreme floods are estimated most often by assuming a hypothetical storm over the drainage basin in question and calculating the runoff from such a storm. The intensity and duration of the storm is based upon rainfall records within the basin, or elsewhere if the basin in question has not experienced severe flood-producing storms. The flood runoff is estimated by applying a runoff coefficient to the rainfall and distributing the total flood over a period of several hours or days, according to the unit hydrograph for the point under study. The unit hydrograph shows the normal distribution of stream flow following a heavy rainstorm, above the base flow due to ground water infiltration, etc. The shape of the hydrographs is reasonably constant for a given stream, regardless of the magnitude of the flood, and a normal or average hydrograph can be prepared from stream flow records during several storms of less than maximum intensity.

Peak floods are frequently greater than the maximum 24-hour flows recorded in stream flow tables. Continuous records are most satisfactory but too expensive for installation at all gaging stations. Continuous records are particularly important in flood-routing studies and in determining the most efficient operation of reservoirs. The time of peak flows may be as important as the magnitude, especially in making plans to regulate peak flows from two or more tributaries, so that they will not coincide further downstream.

The study of floods is more closely related to rainfall than studies of low flows and storage. For large drainage areas stream gaging stations and several rainfall stations are usually available. All too often the rainfall gages are concentrated in the more populated areas in the valleys and no data are available in the upland areas where the rainfall is greatest. The correct determination of average rainfall over the whole area is difficult. On small drainage areas rainfall stations are frequently lacking altogether.

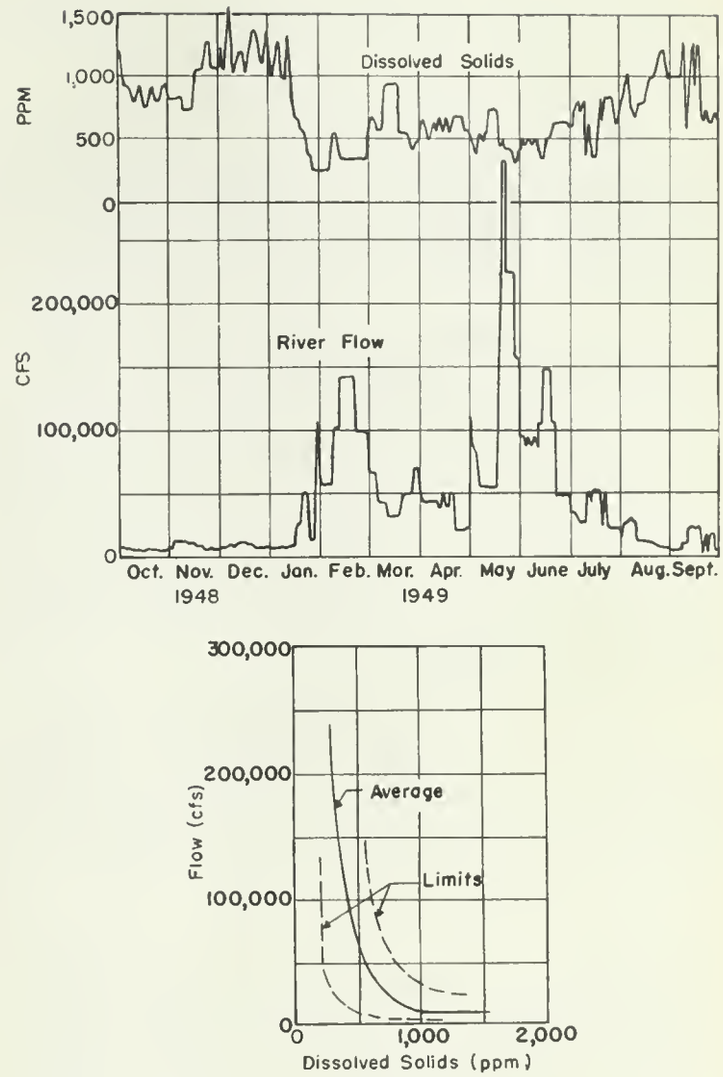


FIG. 38. —RELATION BETWEEN DISSOLVED SOLIDS AND FLOW.

Arkansas River at Van Buren, Arkansas, 1949 (from unpublished records of U. S. G. S.).
Mean flow 1949 = 45, 180 c. f. s.
Mean flow 1928-47 = 32, 330 c. f. s.

CONCLUSION

In closing it is important to emphasize the need for continuance of the stream-gaging program in the United States. Particular attention should be given to stream-flow gaging on small undeveloped rivers in all parts of the country so that the basic data needed for upstream developments will be available. There is always enough interest in the highly developed river basins or in areas where flood problems are severe to assure continuance of gaging there. The lesser streams are more likely to be neglected. The stream-gaging program is costly and we should be sure that the most valuable data are being obtained and recorded, in the most useful form.

It is hoped that efforts to determine the basic relationships and workable general rules of stream-flow phenomena will not stop simply because the availability of data has seemingly eliminated the need in many areas. Hydrological analyses are time-consuming and costly and frequently of questionable accuracy. Short cuts in analysis and guides to normal stream-flow behavior can still play a useful part. The development of a complete understanding of stream-flow behavior will undoubtedly require a greater knowledge of meteorology and its relation to runoff.

DISCUSSION

E. F. BRATER.*—In preparing a paper on such a broad subject as "The Analysis and Use of Surface Water Data," Mr. Hazen was faced with a very difficult task in keeping the length of his presentation within the half hour limit. It seems to me that the scope covered by this title requires a discussion of long term average discharges, minimum flows, and peak discharges. Furthermore, the subject of peak discharges might well be subdivided into large drainage basins, small drainage basins and runoff plots. Mr. Hazen has given an excellent presentation of the hydrological problems of average flows and minimum flows, but he did not have time to go into the subject of peak flows to any extent. I would like to use my discussion period to mention a few items in regard to this latter phase of the topic.

I would first like to speak about small watersheds. By these, I mean watersheds varying in area from an acre or two to as much as ten square miles. They represent an important phase of hydrology because storm runoff from such areas controls the design of small bridges, culverts, and storm sewers. During the past 15 or 20 years, the United States Forest Service, the Soil Conservation Service and other federal and state agencies have obtained rainfall and runoff records from small basins. These records are not only useful for solving specific design problems, but they are supplying the information which is needed to obtain a better understanding of the fundamental principles of hydrology. They are providing us with basic information on infiltration capacity, water losses and runoff. As a result of studies based on such data, dependable methods of predicting peak runoff from small basins are being developed. Records from small watersheds are more useful for basic research purposes than those from large basins because information on precipitation and watershed characteristics can be obtained more easily and accurately from small basins.

In his brief discussion of peak flows, Mr. Hazen suggested using a runoff coefficient to determine the portion of a rain which becomes surface runoff. It seems to me that one of the most important advances in hydrology in recent years is Horton's infiltration theory. Horton pointed out that the soil in any particular condition could absorb water at a certain rate. This rate is called the infiltration capacity (f). If at any time the rate of rainfall (p) is greater than the infiltration capacity, then the

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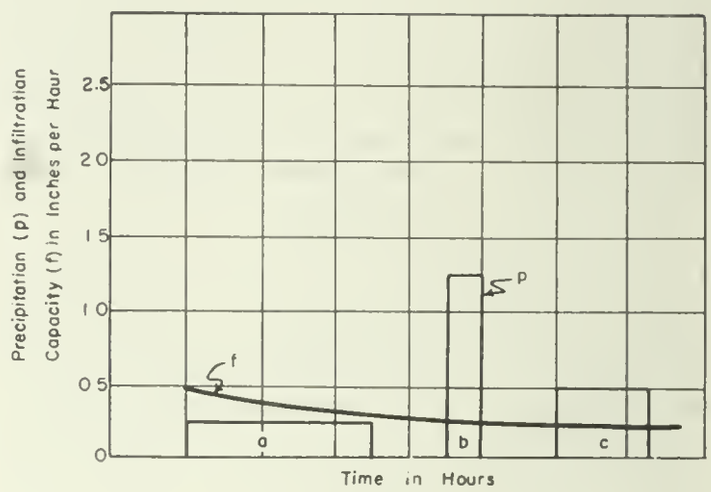

FIG. 39.

excess precipitation (p_e) gathers on top of the ground and becomes surface runoff. This is illustrated in Fig. 39 where rain (a) produces no surface runoff, whereas rains (b) and (c) produce a volume of surface runoff approximately equal to the area of the precipitation graph above the f line. Values of infiltration capacity depend only upon the condition at the ground surface, whereas runoff coefficients vary with the nature of the rain. For example, in Fig. 39 are shown three rains, each having the same total rain, but varying in intensity. The amount of surface runoff, as represented by the area above the f curve, is zero for rain (a), approximately 1.0 inch for rain (b), and 0.63 inch in the case of rain (c). The runoff coefficients would therefore be zero, 80% and 50%, respectively, under identical soil conditions. On the basis of this hypothetical case, one can see that runoff coefficients are likely to be much more variable than infiltration capacities. For example, for nine flood-producing rains on a 4600-acre watershed, located in southeastern Ohio, the infiltration capacity varied from 0.9 to 2.4 inches per hour, a ratio of maximum to minimum of 2.6, whereas the runoff coefficients varied from 34 percent to 5 percent, a ratio of maximum to minimum of 6.8.

W. D. MITCHELL.*—Almost a quarter of a century ago, Mr. Allen Hazen was just completing his now well-known book on "Flood Flows." Over near the end of that book he tucked away a little philosophical observation which recurs to us every

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time we look at a daily record of stream flow: "There is nothing more discouraging than a great mass of data that has not been arranged and studied with reference to the problem at hand." Although stream-flow records are, in general, deplorably short, many now include a mass of data so large as to be downright discouraging. We are grateful to Mr. Richard Hazen for presenting to us this afternoon methods of arrangement and study which should dispel this fog of discouragement.

There is neither time nor need, at this point, to recount all the outstanding features of Mr. Hazen's paper. In the brief time to which these remarks must be confined, it seems appropriate to select a few points with which we have had some experience, and subject them to somewhat more detailed observation.

In the early part of his paper, Mr. Hazen mentioned the duration curve. We should like to emphasize his statement that, among other things, such a curve shows the runoff characteristics of the stream. How true this is will be observed in Figure 40, showing duration curves for two streams: Kankakee River at Momence, Illinois, and Cache River at Forman, Illinois. Momence is in northeastern Illinois, about 30 miles south of Chicago, where the Kankakee River drains a very flat area of deep glaciated soil, whereas Forman is in extreme southern Illinois, near Cairo, where the Cache River drains an undulating area of thin soils which are underlain by tight clay. Each of the curves* is quite typical of the general area from which it has been selected.

It will be noticed that the ordinates are second-feet per square mile, thus minimizing the effect of difference in area. Furthermore, both the curves are for the same 25-year period of record, which tends to minimize the effect of unusual meteorological events. Yet the curves are vastly different. The comparatively flat slope of the Momence curve emphasizes the stabilizing effect of the large natural storage in the basin; the steep slope of the Forman curve reflects the flashy nature of the runoff which, in turn, arises from the rolling terrain, the tightness of the soils, and the lack of sustaining groundwater reservoirs. Another significant feature of the Forman curve is the tendency to break to the left at the upper end. This is not unusual; in fact, the curves for most streams in southern Illinois break more sharply than this one. The cause of the break undoubtedly is associated with the fact that, in contrast to the situation in northern Illinois, most of the streams in southern Illinois have extremely wide flood plains; the shape of the upper end of the curve obviously is an indication of the extent of flood-plain storage.

*"Water-Supply Characteristics of Illinois Streams," p. 70 and p. 277.

It should be pointed out that the duration curve for any stream, if derived from a short period of record, may differ appreciably from that obtained from a long period of record. However, the improvements in hydrologic technique now make it possible to estimate the long-term duration curve from a short-term record; in fact, under favorable circumstances the long-term duration curve may be quite satisfactorily determined on the basis of only miscellaneous low-water discharge measurements.

Mr. Hazen rightfully points out that, "The mean runoff of a stream is its most important characteristic." We would add the thought that, in some instances, a reliable value of mean runoff may be difficult to obtain. If it is to be computed from available record, care must be used that the record is of such length as to iron out the effects of unusual meteorological events. Obviously the mean computed from a few years of severe drought will differ materially from that computed from a short period which includes major floods. As the record becomes longer, the effect of such factors will become less, but one may not wish to wait for a record to grow long before determining a long-term mean. One solution to this problem takes us back again to the duration curve. The area under that curve, which may readily be computed by step methods

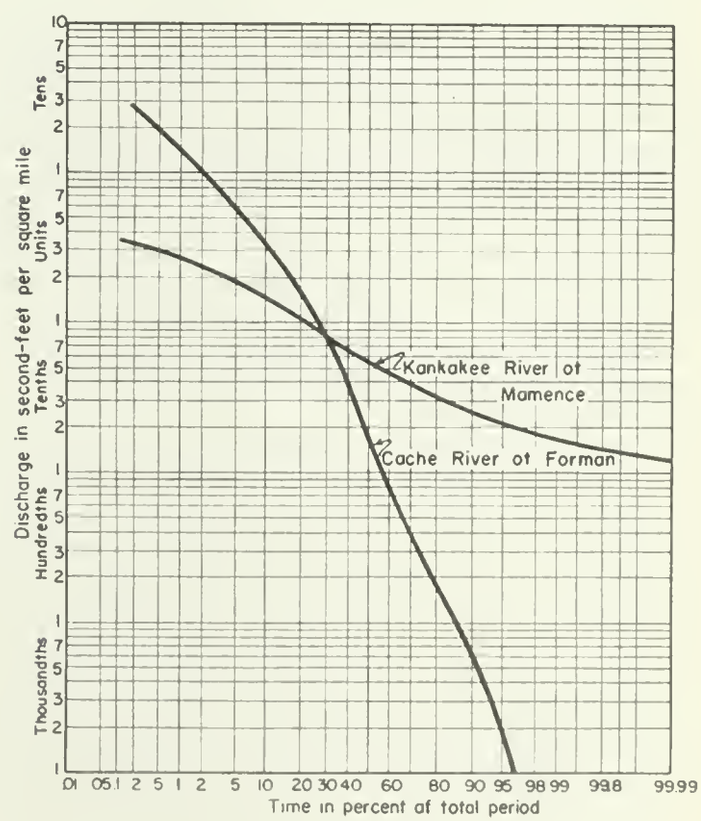


FIG. 40.

of integration, represents the total discharge for the period on which the curve was based. Thus the long-term duration curve, determined from short-term record by methods already mentioned, may be integrated to obtain the corresponding long-term discharge.

As our closing point we should like to refer to (and applaud) Mr. Hazen's plea for particular attention to records at the right place, stream gaging on the small undeveloped streams. That this need is being generally recognized is evidenced by the fact that the Geological Survey, in cooperation with other interested agencies, has greatly expanded—and still is expanding—the number of active gaging stations on the smaller streams. In Illinois, for example, 57 of the present gaging stations have drainage areas of less than 100 square miles. One half of these have drainage areas of less than 20 square miles. On the other hand, however great may be the number of stations, we never can hope to develop a network such as to provide an actual stream-flow record for every site at which stream development may be considered. To meet this situation there must be a continuation of studies which have as their objective the improvement of techniques for transferring data with respect to place, as well as with respect to time. Such studies, in themselves, are a fundamental part of the analysis and use of surface water data.

C. V. YOUNGQUIST.*—The text for my discussion is that of Lord Kelvin who stated, "Nearly all the grandest discoveries of science have been but the reward of accurate measurement and patient, long-continued labor in the minute sifting of numerical results."

It should be noticed that Lord Kelvin placed equal importance on "minute sifting" as on "accurate measurement."

In the field of surface-water hydrology much less effort has been spent on "sifting" or "analysis" than on data collection.

There are at least two cogent reasons for analysis and interpretation to be carried on concurrently with data collection. First—subsequent analysis of data may indicate that important correlative data should have been collected which is forever lost. Second—analysis and interpretation may indicate the lack of need for some basic data collection.

The experience of Ohio may be typical of the evolution occurring in the field of basic data collection. A systematic program of stream gaging was begun in 1921. For 20 years the work was confined largely to data collection with no attempt at interpretation by the State sponsoring agency. The United States Geological Survey had a program of interpretation set up in 1934 as a work-relief project in which some Ohio records were used. Also the flood of 1935 in the Muskingum Basin and those of 1937 resulted in some interpretive flood reports.

In 1940 students under the National Youth Administration program were used to prepare duration tables for selected streams.

In 1941, due to reports of falling water tables over the state, the legislature created the Ohio Water Supply Board. The program was inadequately financed and most of the effort was devoted to reconnaissance surveys of the ground-water situation.

The legislature created in 1945 the Ohio Water Resources Board with broad directives in the field of water resources.

The first interpretive report was that of flood frequency analysis. This bulletin was received well and apparently served a long felt need. It became apparent that there was a need for more flood data which was not obtainable from the regular stream-gaging program.

A system was set up to make field surveys of every local, damaging flood with interpretations as to the cause, frequency, flood discharges, etc. A body of information and data is becoming available which will be of great value in the design of works, particularly on small streams.

A series of two intensive studies was made on Ohio Stream Flow Characteristics, one dealing with flow duration and one water supply and storage requirements. The duration curves were prepared by half decade intervals to facilitate comparative studies of records of varying length. The 90% flow duration value was used as an index in comparing streams and the influence of surficial deposits on stream flow.

In the field of the chemistry of surface waters only superficial work has been done. Much more basic data collection must be carried on. A fertile field for interpretation is in the field of suspended sediment. Since this program is in its infancy much more data collection must precede interpretation.

In all county water resources investigations the characteristics of all the streams in the county are ascertained by miscellaneous discharge measurements made, if possible, at approximately the 90% flow duration period.

The entire program of collection and interpretation of water resources data is carried on in cooperation with the U. S. Geological Survey. Since all activity on water resources in Ohio is consolidated in the Division of Water, Ohio Department of Natural Resources, that agency is the cooperating agency with the U.S. G. S.

*Chief, Division of Water, Ohio Department of Natural Resources, Columbus, Ohio.

ENGINEERING METEOROLOGY*

BY STIFEL W. JENS**

WITH DISCUSSIONS BY GEORGE S. BENTON, IVAN E. HOUK AND PHILLIP LIGHT

INTRODUCTION

It belabors the obvious to do more than indicate the vast areas of engineering in which knowledge of atmospheric temperatures, precipitation (rain, snow, sleet, hail, etc.) and winds is essential to intelligent design and operation of engineering works. Waterway openings in bridges and culverts, urban storm drainage, municipal water supply, irrigation, hydroelectric power, flood control, navigation, erosion control, pollution abatement, structural provision for wind forces and temperature variations, evapo-transpiration in agronomy, and many other applications—each readily brings to mind the engineer's dependence upon and need for meteorologic data and an understanding of the phenomena out of which came such information.

Within the limits of this paper it is impossible to discuss adequately the detailed meteorologic information which the hydraulic or other engineer dealing with water problems should possess; this is available in various books on meteorology as well as in the several works on hydrology. Consequently, the discussion to follow will be confined to rainfall, since it is the atmospheric phenomenon which engages major interest of the engineer because of his concern with surface and ground waters which are residuals of precipitation. There will first be a brief treatment of the causes and types of storm, available data and its limitations, and the processing of rainfall data for engineering purposes. Following this, there will be examples of the use of rainfall information for determining the maximum spillway flow and the minimum yield for a large water supply impoundment.

PRECIPITATION

Causes. All precipitation is the result of the atmosphere's water vapor being condensed, in whole or in part, by cooling. Such cooling of masses of air, which results in fine droplets (clouds or fog) if the point of saturation is reached, can be brought about by lifting or radiation. The latter produces frost or dew, neither of which are of major engineering significance, although the location of foundations and pipe lines always reflects the penetration of frost, and dew must sometimes be given consideration as a contributing factor in corrosion.

Types of Storms. Lifting of vapor-laden air masses can result from (1) topographic influences, (2) convergence or frontal action, (3) thermal convection, or (4) combinations of any or all of these factors. The storms due to the first of these influences, which are more generally termed orographic, are typified by the rise in moisture-laden masses of air as they are forced over mountain barriers or escarpments; e.g., the Pacific Ocean air masses lose their moisture quickly as they rise over the coastal mountains. Frontal type storms occur when a lighter, warm, moisture-bearing air mass encounters a heavier cold air mass and is forced upward and over the latter, giving up its moisture. The entire United States east of the Rocky Mountains has as the major cause of precipitation, convergence and frontal action, typified by the warm, moist air from the Gulf moving northwardly and encountering the southward or eastward moving cold air masses from Canada, or the northern Pacific Ocean. Thermal convection manifests itself mostly in thunderstorms and is caused by localized temperature inequalities in the layers of air immediately above the earth's surface.

The engineer is interested in the fact that storms of wide extent (occasionally covering tens or hundreds of thousands square miles) are frontal or orographic in type and usually are characterized by rather uniform distribution of rainfall, with a relatively small range of intensities. The large watersheds usually involved in major projects for irrigation, land drainage, hydroelectric developments, river navigation and flood control, are concerned with such widespread storms.

Storms characterized by spotty distribution, with a wide range in intensities over a limited area of a few hundred or thousand square miles, result primarily from the convective process. The thunderstorms or cloudbursts are usually of short duration, with very high rainfall rates over an area of not more than a few square miles. The design of urban and airport storm sewers, highway and railroad culverts is usually predicated upon short duration—high intensity precipitation, with a maximum

*Some notes on the Engineering Understanding and Use of Rainfall Data.

**Consulting Engineer, St. Louis, Missouri.

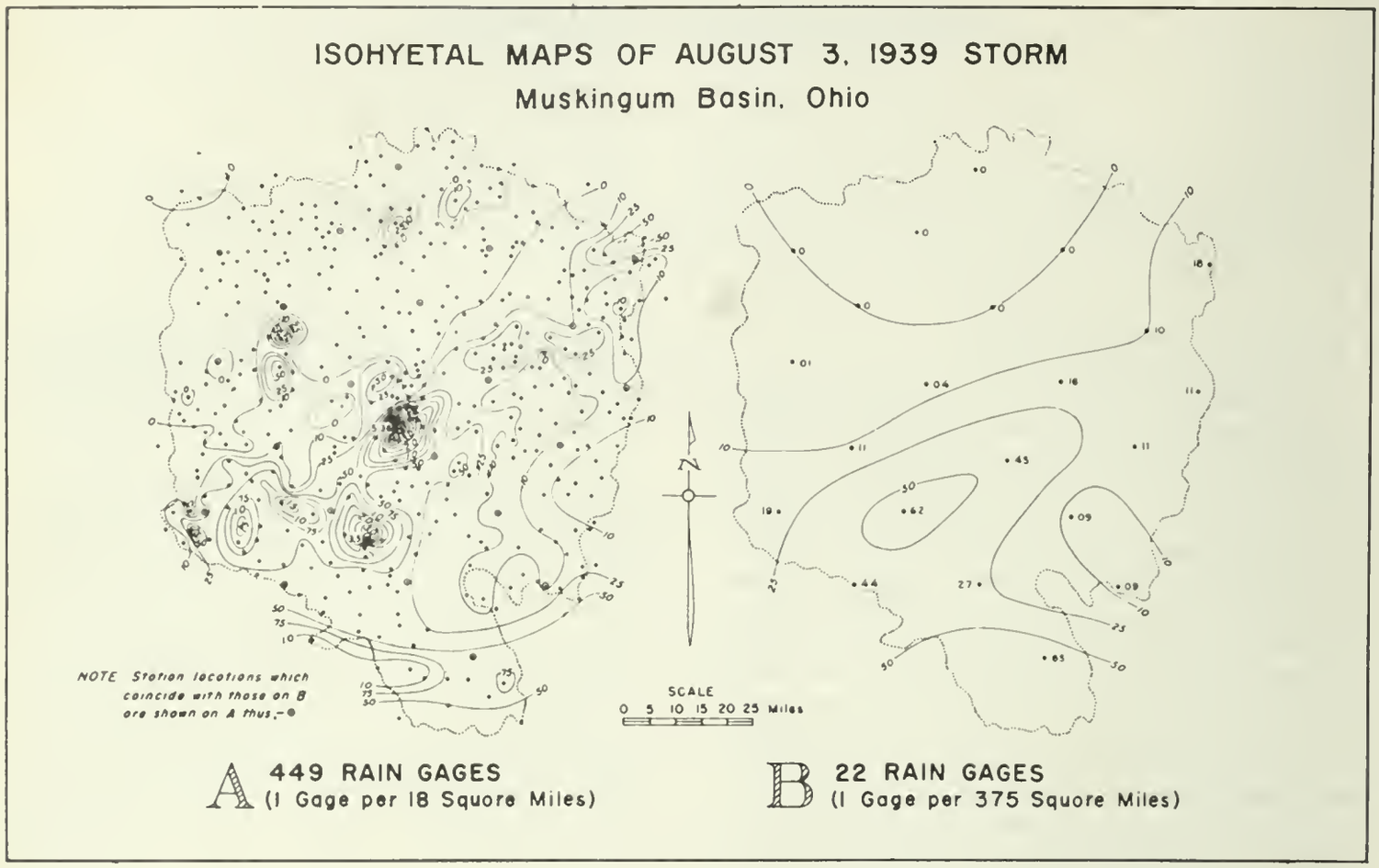


FIG. 41.

time of about 120 minutes.

The hydrologic and meteorologic literature related to the very large area storms is much greater in extent and usefulness to the engineer than is the comparable published information with respect to the local, high intensity rainfalls. For runoff problems, it can be seen that the engineer requires precipitation data for durations closely related to the "time-of-concentration" as that term is used in the so-called "rational" method. For any particular problem, therefore, the type of meteorological data needed is dictated by the time factor of the problem.

Available Data. The literature is ample with respect to the devices and methods for collecting data, and there will be indicated here only the major sources of the data. There are published by the U. S. Weather Bureau each month in "Climatological Data" daily and hourly rainfall amounts from about 6500 stations, of which some 300 are first-order stations, which latter operate recording rainfall gages and make hourly observations of temperature, wind direction and velocity, relative humidity, barometric pressure, sky conditions, and many other meteorological phenomena. The nonrecording stations with cooperative observers make daily observations of precipitation and temperature. Supplementing these data, there were published from 1940 to 1948 by the Weather Bureau, monthly "Hydrologic Bulletins" in which hourly rainfall amounts were given from about 2800 recording gages, together with daily totals from approximately 1600 nonrecording rain gages. Additional unpublished data are regularly collected by various Federal, State and private organizations. Rainfall data cover periods of one to over 100 years, with more than 6000 stations having produced records of 30 to 50 years, which is considered an acceptable length of time for statistical analysis. More complete summaries of available hydrologic data, including precipitation are given in references 18* and 19.

Limitations of Rainfall Data. 1. Gage Density: Comparison of the catch of gages of various diameters disclosed no significant difference (Ref. 3). The sample caught is extremely small when compared to the area it is supposed to represent (average

*See BIBLIOGRAPHY at end of this article for all references.

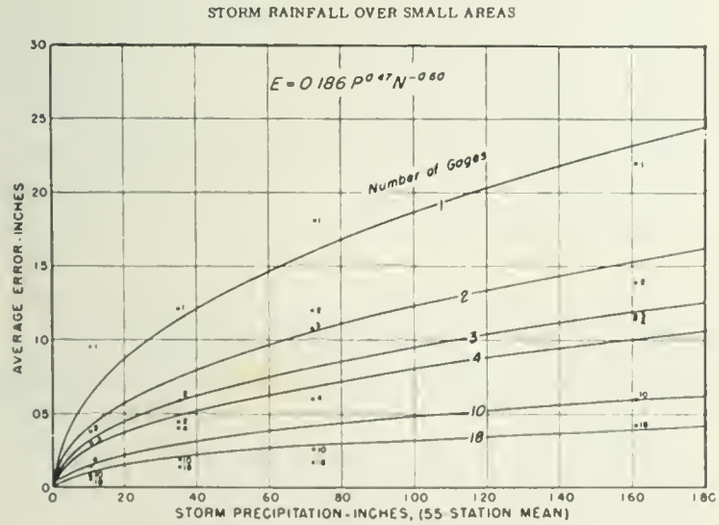
area per gage in 1949 was 375 square miles), but so long as the present methods of measuring rainfall are used, the only modification that can be made is the provision of more gages. Perhaps the new tool, radar, may ultimately be used to gain more detailed knowledge of the areal distribution as well as the representativeness of point-rainfall measurements. All of our existing rainfall data obtained with conventional "standard" rain gages is point rainfall, and this fact must always be borne in mind.

That the provision of more closely spaced gages would be valuable is clearly shown in Fig. 41 (Fig. 1 in Ref. 1). A recent paper (Ref. 2) covers analyses of the data from 55 recording rain gages on about two mile centers, covering an experimental area 10 miles wide by 22 miles long immediately south of Wilmington in central Ohio. The data for April 29 to September 21, 1947, and February 5 to September 22, 1948, indicated 68 storms varying from 0.01 to 4.65 inches, with a maximum duration of 60 hours. Fig. 42 from this report (Fig. 2 in Ref. 2) shows the relative reliability of rainfall computed from networks of differing density. This suggests interestingly that about one-third of the total gages would result in average deviations of from 8% to less than 3% of the catch measured by 55 gages. There seem to be economic optimum rain gage densities. This study also investigated: "the variation in the reliability of a single gage with its distance from the center of the area . . . ; the frequency distribution of the error in assuming a single gage, randomly placed to represent the average within the 220-square-mile area; and the representativeness of a single gage centrally located in areas of various sizes." Space precludes further discussion here, and those interested are referred to the published report (2).

2. Precipitation Measurement Errors: Observational errors are usually small but cumulative, except for erroneous scale readings which although sometimes large, are ordinarily compensatory. Instruments are subject to errors resulting from poor maintenance such as dirt, faulty leveling, corrosion; or limitations of design, such as the inability of the tipping bucket gage to keep up with intense rates, and its dependency on an electric current. Under-registration is corrected on the chart by prorating to the high intensities the difference between the stick measurement of the gage catch and the tipping bucket record. The weighing type recording gage is entirely mechanical, and automatically weighs and records any snow catch as well as rainfall; occasional difficulty arises from the reversing mechanism, and temperature effects on the spring balance.

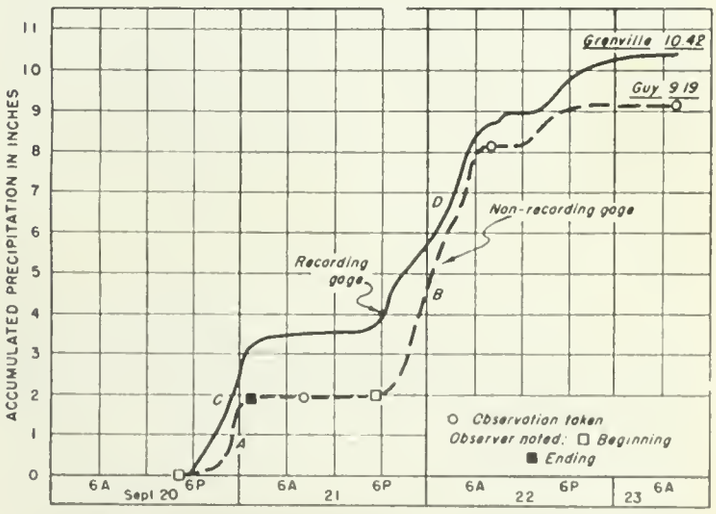
Errors in the record data due to faulty gage exposure, and changes in location of the gage during the period of record can be detected and should be corrected before the data are used for statistical purposes. Improved correlation between stream flow and precipitation can be obtained after such adjustment of the rainfall data. The scheme of double mass plotting and adjustment of the data is described in detail in Reference 3. Essentially, the method depends on the fact that if the entire record at a single station has been observed under the same conditions at the same location, its accumulated precipitation plotted against the mean accumulated precipitation for a large group of stations in the same area results in an unbroken straight line. Changes in slope of the line indicate changed location or conditions of observation.

3. Cooperative Observers Data: The bulk of published daily rainfall records are from measurements taken by cooperative observers with standard nonrecording rain gages. The time of observation varies, and the recorded amount is usually for the preceding 24 hours. Some observers note the beginning and ending of rainfall and consequently a more realistic average intensity-duration value can be obtained, than the average rate for 24 hours, which is all that can be assumed for those records with no qualifying notes. The date of the recorded 24-hour total is sometimes erroneous, where, for example, all rain occurred before midnight but was measured the following morning. Careful com-



--Relation between storm precipitation, number of gages in 220 sq mi, and the error of computed average precipitation

FIG. 42.



—Mass Curves for Grenville, N. Mex., and Guy, N. Mex., September 20-23, 1941

FIG. 43.

parison with nearby recording gage records, particularly by the use of mass curves can frequently guide judgment in the proper interpretation of cooperative observers records, as is shown in Figure 43 (Fig. 3 in Ref. 3).

Some engineers (Ref. 4, p. 370; Ref. 17, p. 80) believe it desirable to attempt to have all rain gages read at the same time of day. Currently published records of recording gages give the midnight readings; cooperative nonrecording gages are read variously in the morning, at noon, or in late afternoon or evening. This standardization of time of observation appears to be particularly desirable in light of the fact that "analyses of a large number of major storms ... shows that the maximum rainfall intensities usually occur in periods of 12 hours or less, interspersed with periods of minor rainfall rates. It is considered essential therefore, that maximum rainfall quantities be determined for periods of 6 hours and in some cases for shorter time intervals." (Ref. 4, p. 371.) The Hydrometeorological Section of the Weather Bureau, however, has "found that the staggered hours of observation have proved blessings in disguise, disclosing as they do the times of beginning and ending of rain." (Ref. 4, p. 382.)

4. Published Rainfall Data: The hourly rainfall amounts given in the monthly "Hydrologic Bulletins" and in "Climatological Data" represent the precipitation that fell in the clock hours shown as taken from the recording gage charts. Rainfall rates for periods of less than 1 hour and maximum intensities for 1, 2, 3, or more hours, where these do not coincide with clock hours, cannot be obtained from the published records; the original records must be examined to obtain these.

Excessive precipitation for various durations from 5 to 80 minutes for all data subsequent to March 1934 is defined by the Federal Weather Bureau as the rates equal to or exceeding those in Table I. For the states of North Carolina, South Carolina, Georgia, Florida, Alabama, Mississippi, Tennessee, Arkansas, Louisiana, Texas and Oklahoma, the amount A in hundredths of an inch falling in time t in minutes is: $A = 2t + 30$; for all other states, $A = t + 20$ defines the excessive rates. In the table these have been converted to inches per hour. The Weather Bureau rules further require that even where the excessive rate does not continue for 60 minutes, when the rain equals or exceeds the rates in Table I, all tabulations must show the accumulations for 60, 80, 100, and 120 minutes.

Prior to 1934, any precipitation occurring within the 120-minute period but after termination of the excessive rate, was not shown, and the criteria were different. These earlier data in the Annual Reports of the Chief of the Weather Bureau, for the "60 minute and 120 minute excessive rates determined from the published data are from 5 to 10% low. The published data also do not give the actual maximum rates of precipitation during 5 minute intervals at any point of the storm, but only the maximum rate of precipitation during the particular 5 minute increments of time shown in the table of accumulated amounts. In consequence the 5 minute excessive rates determined from the published data are also about 10% low. Intermediate rates are substantially correct, as might be expected." (Ref. 5, p. 165.)

Yarnell's Charts: Despite their limitations (which arise from the data), from the engineer's viewpoint, the 56 isohyetal charts in Reference 6, summarizing the late David L. Yarnell's analyses of all first order Weather Bureau rainfall records through 1933 are the most useful and time-saving meteorological data that have been collected and processed for practical application. These charts indicate the probable frequency of given rates of precipitation for various durations over most of the United States. Values are given for average intensities expected to be equalled or exceeded about once in two years, on the average, for durations of 5, 10, 15, 30, 60 and 120 minutes; for frequencies of about once in 5, 10, 25, 50 and 100 years, values are given for 4, 8, 16 and 24 hours in addition to the shorter durations. These charts "are based upon the weighted rainfall experience of all Weather Bureau stations, and therefore are more dependable for design than the records of any individual station." (Ref. 3, p. 10.) Yarnell did, however, con-

Table I

Inches Per Hour At or Above Which Precipitation May Be Considered Excessive

Storm duration in minutes	5	10	15	20	25	30	35	40	45	50	60	80	100	120
Southern states	4.80	3.00	2.40	2.10	1.92	1.80	1.71	1.65	1.60	1.56	1.50	1.43	1.38	1.35
All other states	3.00	1.80	1.40	1.20	1.08	1.00	0.94	0.90	0.87	0.84	0.80	0.75	0.72	0.70

sider storms to have fallen in the longer times even though they actually terminated at some earlier time. This principle of "extended duration" (Ref. 7, p. 953) in the use of published rainfall data has been generally recognized as valid, and necessary to a proper determination of frequency.

The results of an intensive study (published 1938) of rainfall intensities and frequencies for Chicago based upon 659 station-years, when compared with the values taken from the Yarnell charts, indicated, "The values are almost identical, with the exception of the 100 year, 2 hour rate ...." (Ref. 8, p. 391.)

With almost 18 years additional data available there is a definite need and desirability to revise the Yarnell charts, by adding the very valuable additional data collected since 1933. It is not expected that this would alter substantially the charts for most of the humid area east of the hundredth meridian, except for the mountainous areas of the Appalachians and Alleghenies, but it would be expected to materially improve the charts for the area west of the Rockies. Also, refinement and substantiation would necessarily result from the better data of the past 18 years.

Two earlier point rainfall studies of considerable magnitude covered the records of the eastern United States and were made by Adolph F. Meyer (Ref. 5, p. 158) and Arthur E. Morgan for the Miami Conservancy District in Ohio (Ref. 9). Meyer developed intensity-duration formulas for heavy storms of short duration for several meteorologically homogeneous areas for frequencies of once in 1, 2, 5, 10, 25, 50 and 100 years. Morgan's work resulted in isopluvial charts for storm periods of 1 to 6 days giving the relation of rainfall depth to duration for frequencies of once in 15, 25, 50 and 100 years. The Conservancy District revised Morgan's study, using all data through 1934 and this 1936 edition (Ref. 9b) covers the area east of the 103d meridian with 24 isohyetal maps giving rainfall depths for 1, 2, 3, 4, 5 and 6 day storms for the above-mentioned frequencies.

The Yarnell data and the latest Miami Conservancy information supplement one another, although it should be noted that the latter covers durations beyond those for which point rainfall in itself is useful.

Processing of Rainfall Data for Engineering Purposes. Ideally, the engineer using precipitation data requires knowledge of its intensity-depth-area-duration-frequency characteristics. No solution for this five-dimensional problem has been developed to date, largely because of limitations in the data.

The economical and adequate design of urban, airport, industrial, highway, railway and agricultural drainage facilities, and of water conservation and utilization projects rests on the frequency, intensity, and duration of excessive rainfall rates, and for large developments a knowledge of the area covered by the rainfall.

Where the problems are concerned with the stream-flow of a natural river, gagings of the stream are used where available. Such stream gagings are currently being obtained at more points on more streams, and are becoming of satisfactory length of record for statistical study in constantly greater quantity, but the longer and more prevalent rainfall records always will be heavily used to give primary data for studying stream-flow and runoff. The brief examples of engineering use of rainfall data given later will illustrate this very pointedly.

Point Rainfall Data: The Yarnell charts (Ref. 6) and many thorough local studies of point rainfall data (e.g., Ref. 8, 10, 11) have satisfactorily developed the intensity-duration-frequency of excessive point rainfall for practical purposes. The Yarnell studies cover durations of 5 minutes to 24 hours, but the application of undiminished point rainfall data to areas involving times beyond about 120 minutes is in error because of the known diminution in average rainfall depth as the area of coverage increases.

Station-Year Method. A great many engineering projects involve runoff from areas of less than about 1 square mile, or times of concentration of less than 2 hours, including the bulk of municipal storm drainage facilities, which in the many great population centers aggregate many billions of dollars in cost. Principally because of this latter fact, many statistical studies have been made of the available data on excessive rainfall rates for short intense storms.

Even with respect to these data, however, it was recognized years ago that the excessive precipitation data from a single station were usually inadequate for statistical purposes. Meyer (Ref. 5, p. 158) and the Miami Conservancy District (Ref. 9) initiated what has since become known as the "station-year" method. "The method consists essentially of collecting for an area all the rainfall records of whatever length and treating the sum of all the records for all stations (called the 'station-year record') as if it were a single record for the mid-point of the area under consideration.... The combined experiences of all stations in the area are assumed to give a weighted average which may be regarded as the probable average experience for any one point within that area." (Ref. 10, p. 634.) Two requirements must be met to render the station-year method valid: (1) rainfall characteristics throughout the area must be fairly uniform, or the region must be meteorologically homogeneous (and the frequency determination improves in quality with increasing number and better spacing of sta-

tions); and (2) a satisfactory length of record from which the average was determined. "It is obvious that the reliability of a mean frequency based on data from a single station is a function of the length of record for that station, because the longer the record, the more occurrences there will be upon which to base a frequency calculation. This must be true also for a mean frequency based on the data from several stations in an area. . . ."

Regardless of how many station-years of record are used, there will always be a certain degree of unreliability in the frequency determinations simply because the actual years of record are not sufficient to sample all the possible annual variations in the rainfall characteristics." (Ref. 10, p. 640.) This recent study (Ref. 10) developed a measure of the reliability of frequency-intensity values as determined by the station-year method, and contained and provoked thorough discussion. It was also emphasized how handicapped we are so long as we do not have accurate data upon which to base a study of the areal extent of rainfalls of certain intensity and frequency.

The establishment of dense networks of rain gages such as those above discussed (Ref. 2) in the Muskingum Basin in Ohio (about 500 automatic rain gages within 8,000 sq. mi.), the San Dimas Experimental Area in California, the Susquehanna and Ohio Rivers above Pittsburg, the Tennessee River Valley, and in the Climatic Research Unit of the S. C. S. in Oklahoma will ultimately make possible the determination of the frequency of rainstorms of a given area and intensity.

There was published recently (Ref. 12, p. 347) a study of "Relation between Maximum Observed Point and Areal Rainfall Values" with Fig. 44 (Fig. 4 in Ref. 12) the end product. Most of the available data is between a point and 20,000 sq. mi. and the curves are most reliable in that range.

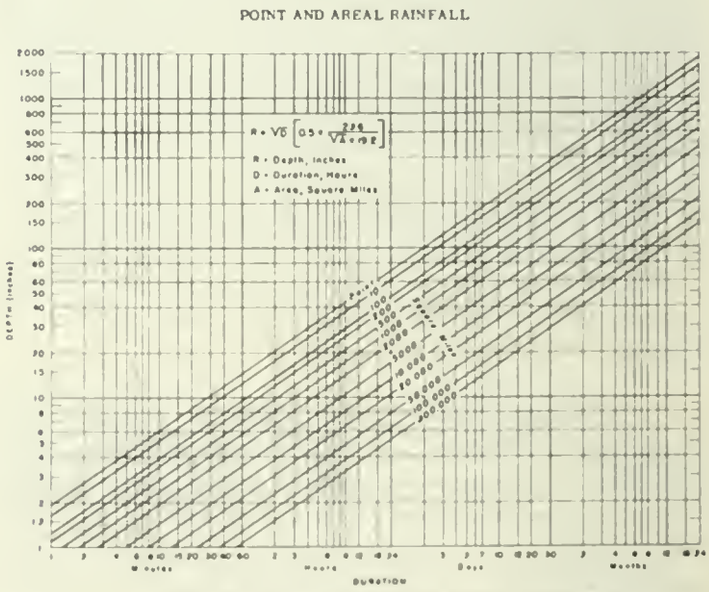
Fig. 45 indicates the variation in average rainfall intensity with area and duration for times of 60 minutes or less as determined in a study (Ref. 13) of data from several rain gages in Boston, in New Orleans and in Cambridge, Ohio.

General Applicability of Point Rainfall Intensity-Duration Data: In a presentation of the principles of design of drainage facilities for military airfields (Ref. 14, p. 698), the following very significant statement is made: "A study of rainfall intensity-frequency data for a large number of precipitation stations indicates that there are fairly consistent relationships between the average intensity of rainfall for a period of 1 hour and the average rates of comparable frequency for shorter intervals, regardless of geographical location of the stations or frequency of 1-hour rainfall." What are termed "standard rainfall intensity-duration curves" were then developed and are herein reproduced as Fig. 46. Each of these curves is assumed to represent average rates of rainfall for the durations indicated, for the same average frequency of occurrence; and as is the case for similar curves developed out of local data, the precipitation rate for any duration is the average rate only, and the curve does not reflect any specific or average storm pattern.

These Corps of Engineers standard rainfall curves are characterized by numbers which correspond to the average rates of rainfall in inches per hour for 60-minute durations. To use them it is necessary to know this hourly rate for the particular frequency desired for the location of the project under design.

When the curves of Fig. 46 were first published, it was pointed out that "the rainfall intensity curves for rainfall on the eastern coast of Australia conform closely with the curves . . ." in Fig. 46 (Ref. 14, p. 791). Subsequent to the preparation of the C. of E. curves, there were plotted rainfall intensity data obtained from Europe and Southeast Asia and there was close conformity to the data plotted for the United States. (Ref. 14, p. 847.)

The study resulting in these "standard rainfall intensity curves" seems to bear very significantly upon the fact that in the station-year method, practical results have been achieved even where the stations may have been quite widely separated. In the studies of intense precipitation at Chicago, Illinois (Ref. 8) gages as far apart as Boston, Mass.; Yankton, S. Dakota; St. Paul, Minn.; and Knoxville, Tenn.; were used, and criticized (Ref. 8, p. 380) but the defense of such use was: "The important implication . . . is that within the high intensity brackets used in these studies, stations many hundred miles apart and with dissimilar annual rainfall



-Depth-area-duration curves enveloping observed rainfall depths over areas between a point and 200,000 sq mi

FIG. 44.

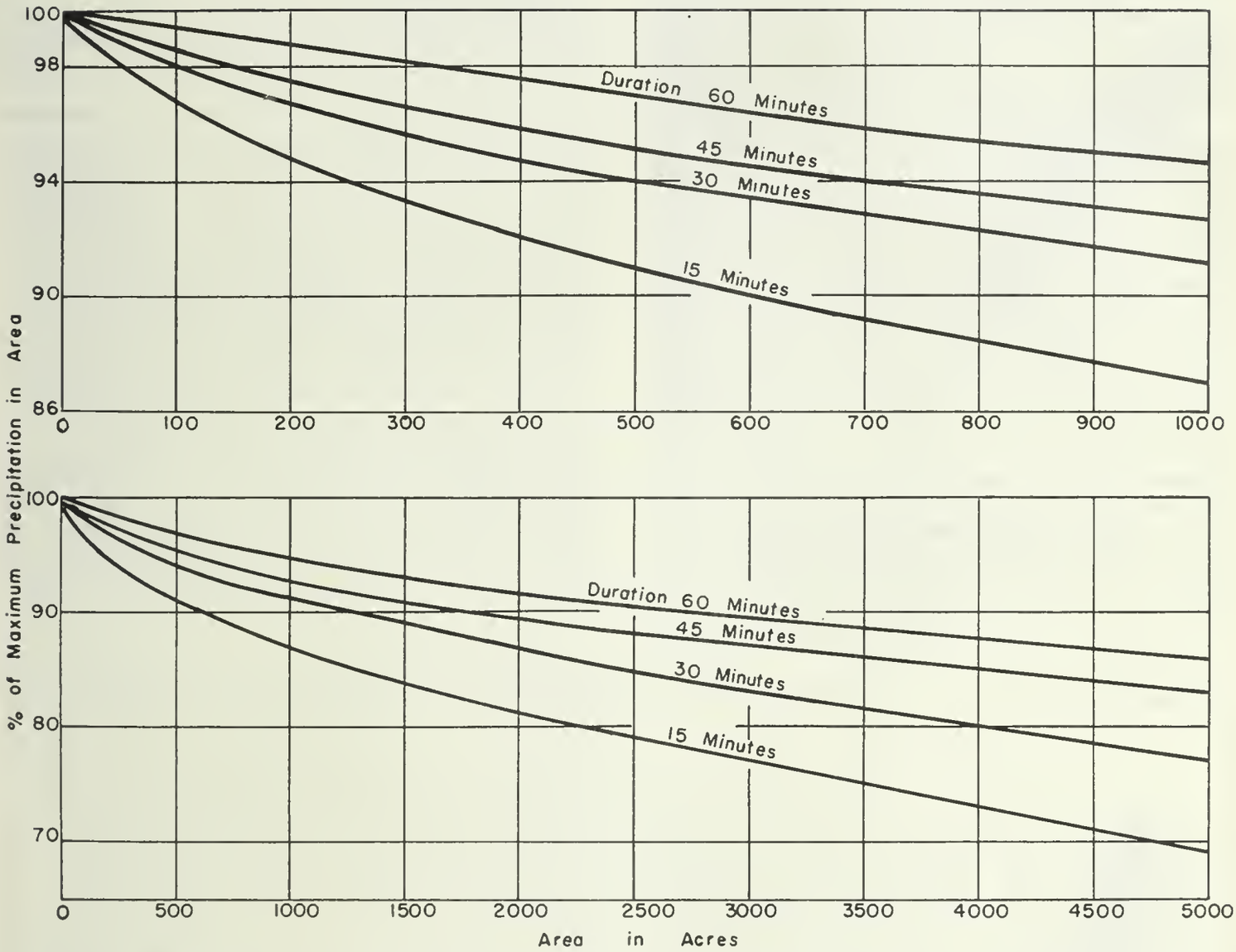


FIG. 45. —PERCENTAGE RATIOS OF AVERAGE INTENSITY OF RAINFALL OVER VARIOUS AREAS TO MAXIMUM WITHIN THE AREA.

Curves based upon storms at Boston, Massachusetts and New Orleans, Louisiana. From data in "The Distribution of Intense Rainfall and Some Other Factors in the Design of Storm-Water Drains," by Frank Marston (Ref. 13).

characteristics have similar frequency-intensity characteristics." (Ref. 8, p. 400.) It would seem that the overwhelming majority of high intensity short duration storms as caught by recording rain gages are of the thermal convective type and apparently the occurrence and variation of rainfall intensities in such storms, both as to rate and duration, is likely to be the same irrespective of geographic location, so long as orographic effects are absent. The considerable difference in average annual number of thunderstorms at widely separated stations (Table 1, p. 3 of Ref. 15) such as New Orleans 74 per year, Boston 18, St. Louis 53, Chicago 40, New York 28, Portland, Oregon 5, does continue some question as to the propriety of the frequency determinations resulting from a combination of data from such far-apart stations. However, statistical checks of the data from 10 stations as used in the Chicago study (Ref. 8, p. 384) "seem to confirm the conclusions that the observed differences between the 10 stations studied in detail are such as might be expected as chance variations in samples of the size observed, if there were no variation in the cause systems affecting the 10 stations."

Intensity-Duration Formulas: As far back as 1891, the late Professor A. N. Talbot (Ref. 8, p. 356) proposed the first rainfall intensity formula for excessive rates for short duration storms. For storms of a given frequency, but not longer than 120 minutes, the rectangular hyperbola $i = \frac{A}{t + b}$ was found to best fit the data. In this, "i" is the rate of

rainfall in inches per hour; "t" is the duration of the excessive rainfall in minutes; "A" and "b" are abstract constants, reflecting meteorological conditions at the specific locality, and are obtained from the observed rainfall data by trial.

Adolph F. Meyer in 1917 published (Ref. 5) the formulas he developed out of a study of excessive short-time precipitation data covering almost 2,000 storms recorded at the Weather Bureau stations from 1896-1914. Stations indicating similar rates were placed in 5 geographic groups and formulas for amounts and rates of various frequencies were set up. The frequencies were those obtained from the combined record of all stations in each group, selecting the rates of rainfall which would probably be exceeded in 1, 2, 5, 10, 25, 50 and 100 years. Table 15 of Ref. 5, p. 196ff. lists "Intense Precipitation Exceeded with Given Frequency as Determined by Meyer Formula," and Table 16 of this same reference compares the various earlier rainfall formulas. Rainfall studies for New York City for 5 to 120 minutes duration (Ref. 11, p. 609) resulted in 6 modified exponential formulas for 1, 2, 5, 10, 25 and 50 year frequencies as best fitting the observed data.

It has generally been found that the Meyer type formulas only fit the observed data for the shorter times, with an exponential formula of the type $i = \frac{C}{t^n}$ more accurately fitting the intensity-duration data for longer times such as are found in the design of agricultural drainage and the analysis of stream flow for small watershed. Concentration times for most systems of storm sewers rarely exceed 180 minutes, whereas the concentration time at the upper end of agricultural drains will vary from 50 to 1200 minutes on watersheds of about 1000 acres, depending on watershed characteristics such as slope, shape and development. And concentration periods of more than 4000 minutes are encountered for drainage areas of 300 square miles. The late Merrill Bernard presented (Ref. 16) a very fine study of intensity formulas for 120 to 6000 minutes and presented charts for the entire part of the United States east of the 101st meridian, on which are given values for the constant "K" and the exponents "x" and "n" in the formula: $i = \frac{KF^x}{t^n}$;

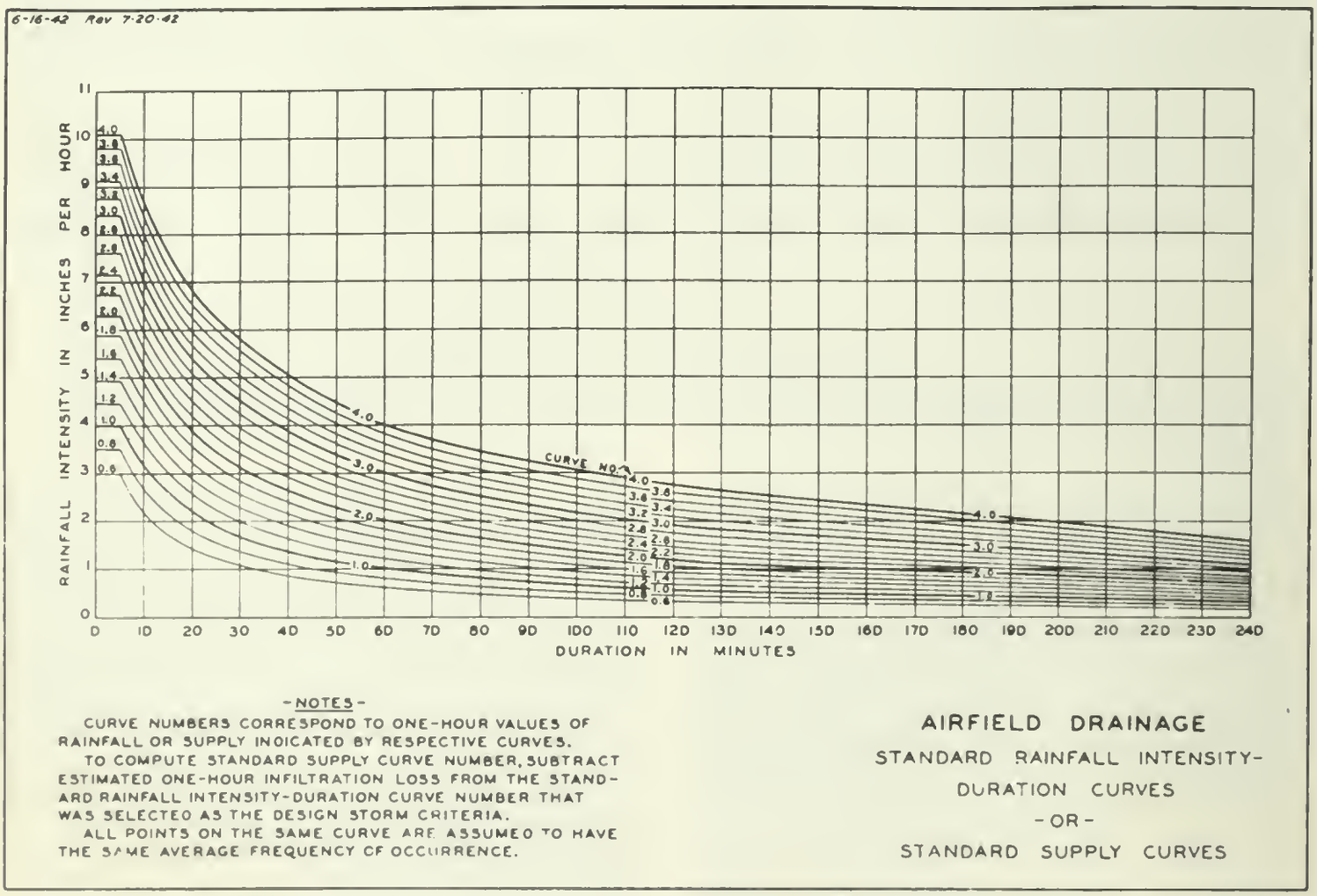


FIG. 46.

i = average rainfall intensity in inches per hour; F = frequency in years; T = duration of rainfall in minutes (not less than 60 minutes); K = coefficient; x and n, exponents depending on locality. A check test (Ref. 16, p. 620-1) indicated that the formula and charts may safely be used for durations considerably longer than the 4 days originally envisioned for the upper limit of the proposed intensity-duration formula.

Summary of Point Rainfall: There are available for the designer the general intensity-duration formulas, curves and charts listed in Table II.

From the earlier discussion of the information in this table, the designer can choose the most applicable formula, curves or charts, supplementing these wherever practicable with a study of the pertinent local data.

Areal Rainfall Data: Point rainfall uninfluenced by orographic considerations is generally over only about 1 square mile as a maximum. Sometimes the limitation is expressed as concentration times of less than 120 minutes. It has been suggested (Ref. 3, p. 20) that 3 additional ranges of storm area be considered: "(2) Small areas (1 sq. mi. to 200 sq. mi.) (3) Intermediate areas (200 sq. mi. to 2000 sq. mi.) (4) Large areas (2000 to 10,000 sq. mi.)." For any of these larger areas for any particular storm the average depth over any specific areas such as a watershed can be obtained by three methods, all using the available data from the gages within and immediately around the area. Thorough discussions have been published (Ref. 17, p. 85; Ref. 18, p. 44; Ref. 19, p. 77; Ref. 3, p. 19) of the relative merits of (1) arithmetic averages of the rain gage amounts; (2) weighting of each gage amount in proportion to the area presumed to be represented by the gage, such proportionate areas being determined by the Theissen method; or (3) by plotting from the rain gage data, contours of equal precipitation or isohyets, and planimetering each contour and determining volumes of rainfall by methods similar to those used in earthwork computations.

For maximum possible precipitation in the United States east of the 105th meridian, there are available charts (Ref. 20) similar to Fig. 47 for 10 sq. mi. and 6, 12 and 24 hours duration; for 200 sq. mi. and 6, 12, 24 and 36 hours; for 500 sq. mi. and 6, 12, 24, 36 and 48 hours. The text of Ref. 20 should be consulted before using these data to understand their limitations and the assumptions used in developing them.

In 1937 there were initiated cooperative studies of 1300 great storms of record in the United States by the Corps of Engineers, the Department of Agriculture, and the Weather Bureau. The late Merrill Bernard in 1944 reported on the results of the analyses of the first 350 of the storms, and in his paper (Ref. 4) he tabled 150 grouped into geographic regions. For each storm there are listed the locality,

GENERALIZED ESTIMATES
MAXIMUM POSSIBLE PRECIPITATION

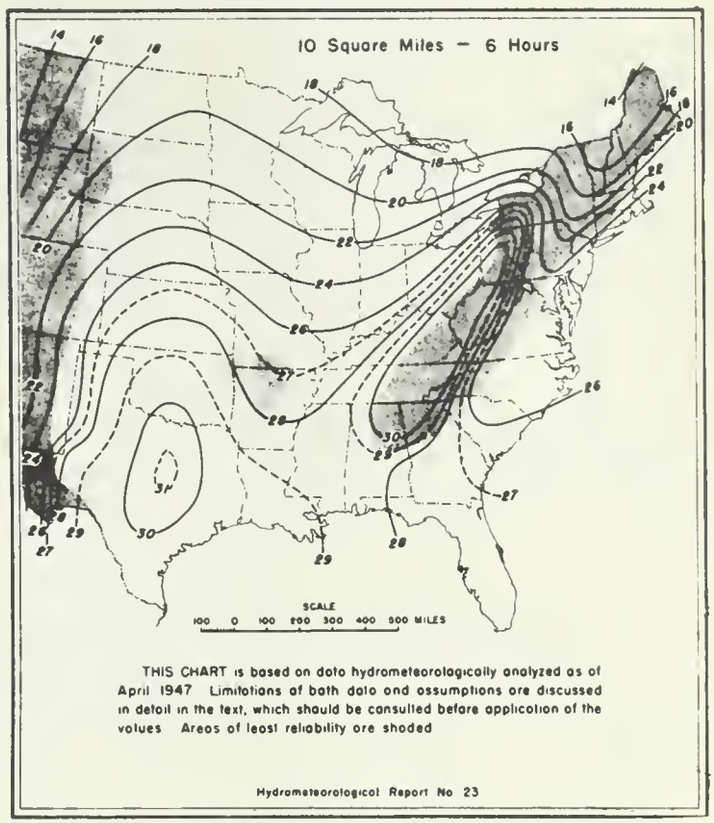


FIG. 47.

center, dates, total storm area, duration and average depth, and average depths for various areas for each 12-hour period of the storm. This study (Ref. 4, p. 334) also describes the methodology of the Hydrometeorological Section of the Weather Bureau in determining the theoretically maximum possible storm rainfall for various areas. A discussion elicited by this paper (Ref. 4, p. 372) describes the additional processing of the data as accomplished by the Corps of Engineers, pointing toward design uses of the processed information.

It has been stated (Ref. 4, p. 380) properly that such maximum possible storms "should not be the basis for design in any except the case where social and psychological considerations dictate security beyond economic justification. In any case, it would seem as much an advantage to the flood control designer to know the upper limit of his major factor, the storm, as it is to the structural designer to know the ultimate strength of his materials."

EXAMPLES OF THE ENGINEERING USE OF RAINFALL DATA

Maximum Runoff: Spillway Design Flood. The water supply impoundment for Houston, Texas, required the determination of the maximum probable

Table II

Summary of Rainfall Intensity-Duration Frequency Formulas, Curves and Charts

Item	1 Locality to which formula, curve or chart applies	2 Formula or chart proposed by	Date	3 Background data	4 Formulas Amt. = I Intensity = i	5 Limits of t, in min.	6 Freq. Once in __ years	7 Remarks
1	United States east of Rockies	A. F. Meyer (Ref. 5)	1917	Excessive precip. 1962 storms at 43 stations 1896-1914 inclusive	$I = \frac{At}{t + b}$ $i = \frac{A't}{t + b'}$	5 to 120 incl.	1, 2, 5, 10, 25, 50, 100	Table 15, Ref 5 gives formulas for 5 geographic groups and for the frequencies noted in column 6.
2	All of United States	David L. Yarnell (Ref. 6)	1935	Excessive precip. from 211 recording gages—all data up to and including 1933	62 isohyetal charts	5, 10, 15 30, 60, 120, 240, 480, 960, & 1440	2, 5, 10, 25, 50, 100	Values can be read directly from charts for any geographic location. Values for far West to be used with considerable caution.
3	United States east of 101st meridian	Merrill Bernard (Ref. 16)	1932	Data of A. F. Meyer and of A. E. Morgan	$i = \frac{KF^x}{t^n}$	120 to 6000-9000 (12 hr. to 6 days)	5, 10, 15, 25, 50, 100	Figs. 18, 19 & 20 of Ref. 16 give values of K, x and n for any location east of 101st meridian.
4	All of United States	Corps of Engineers, U. S. Army (Ref. 14)	1945	Yarnell's data in Ref. 6 and other data	Intensity-duration curves	0 to 240 min.	Values along any 1 curve have same freq.	Requires determination of 60-min. average intensity for the desired frequency at the point under design. Max. 60-min. rate on curves: 4 i. p. h.
5	United States east of 103d meridian	A. E. Morgan Miami (Ohio) Conservancy District Rev. (Ref. 9)	1917 1935	Revised 1936 edition based on all available rain data through 1934	Graphs of time-area-depth curves for northern and southern states	1, 2, 3, 4, 5 and 6 days	12, 25, 50, 100	Isopluvial charts given max. 1, 2, 3, 4, 5 or 6 day rainfall for freq. in column 6; report also gives precip. vs. area for every storm.

spillway flood from the 2840 square miles of the San Jacinto watershed above the dam site a few miles downstream from Huffman, where the main stream (2791 square miles tributary area) has been gaged since October 1936. A somewhat longer stream-flow record starting in October 1928, was available for 1911 square miles of the West Fork near Humble, Texas. The studies herein discussed were made in 1944 (Ref. 21) and utilized the stream-flow records through September, 1943.

A relatively long record of rainfall, with 55 years at three stations in this general region and somewhat shorter but farily satisfactory records at a number of other stations, were available. (See Fig. 48 for location of gages in and around the basin.) The short stream-flow record made it necessary to study the location, extent and intensity of precipitation, and the meteorological conditions attending great storms of record in the general vicinity of the project; and the probability of these great storms occurring over the basin above the dam site; and the probable peak flows resulting therefrom. This discussion is confined to the engineering meteorology. The techniques of determining net effective rainfall and processing it by means of a unit graph necessarily would require another paper.

The Huffman stream-flow record along with the rainfall records indicated six hydrograph rises for detailed study—those of November and December, 1940, April and October 1941, April 1942 and July 1943.. Mass rainfall curves were developed for each rainfall station in and around the basin for each of the above six storms, primarily to secure an indication of time of excess rainfall to be used in developing unit graphs.

For the May 1929 and November 1940 storms, the United States Geological Survey from high water marks had estimated the highest record peak flows at Huffman (also applicable to the dam site) as 237,000 and 253,000 c.f.s. respectively. For the November 1940 storm, 12-hour rainfall amounts obtained from isohyetal maps (similar to Fig. 48) were apportioned into hourly amounts based upon consideration of the hourly distribution of the rainfall as given by the recording gages nearest the basin. Total equivalent uniform rainfall depth over the 2800 sq. mi. was 11.7 inches. This detailed

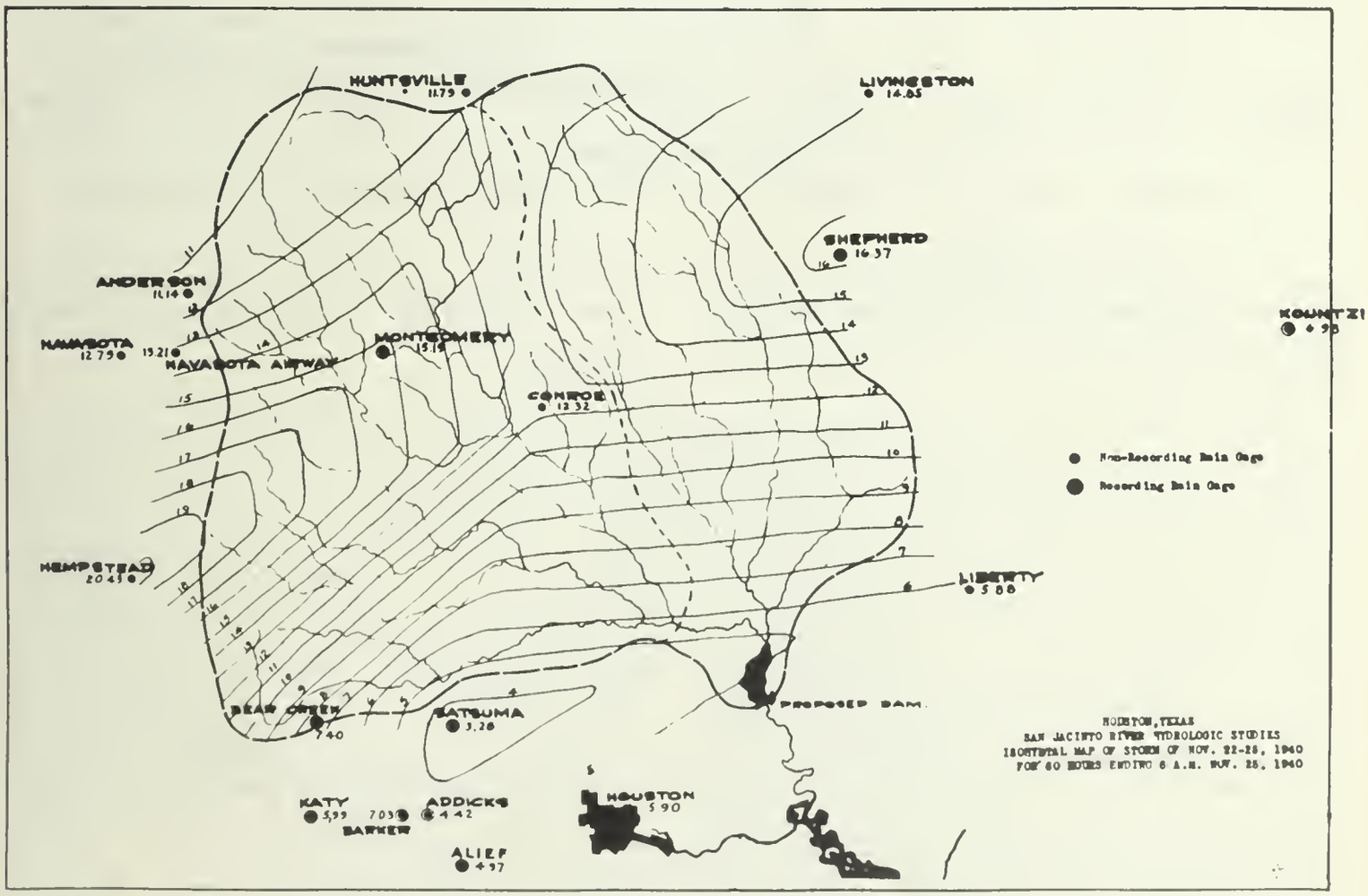


FIG. 48.

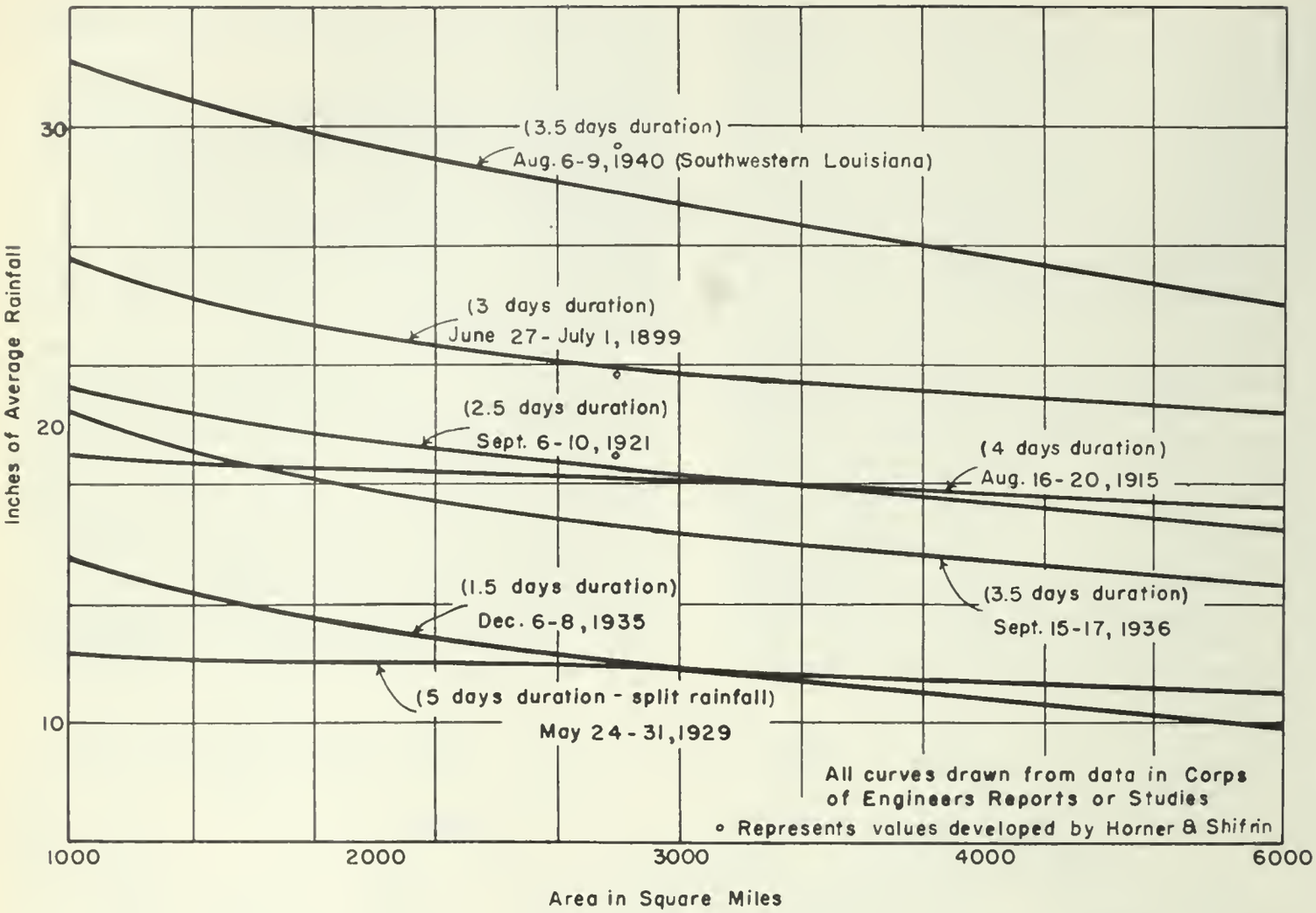


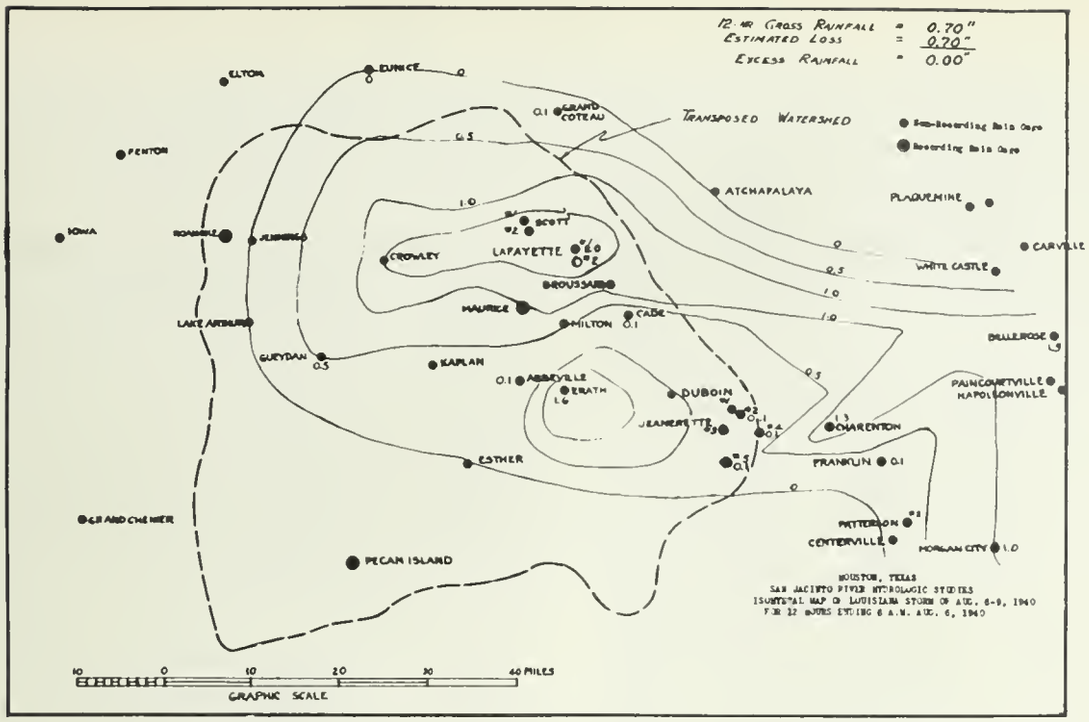
FIG. 49. —SAN JACINTO RIVER HYDROLOGIC STUDIES, DEPTH-AREA DATA, TEXAS STORMS.

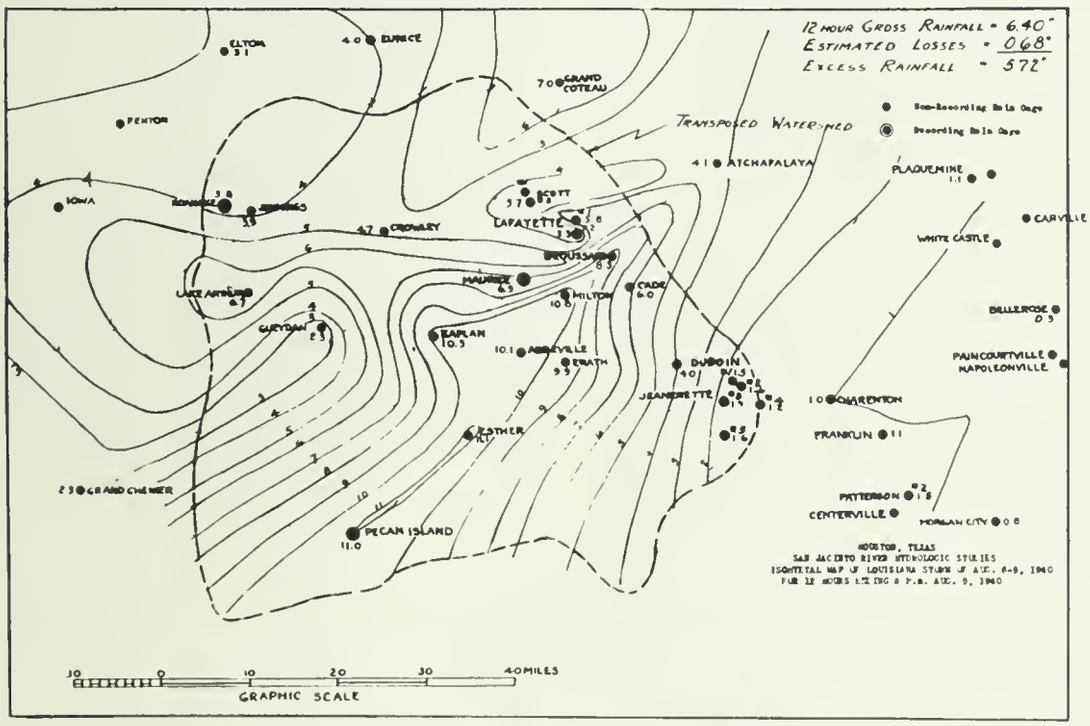
study of the storm causing the greatest flood flow of record was undertaken to develop the best possible techniques for translating the rainfall of a great storm into stream flow for this particular watershed.

Fig. 49 gives area-depth curves for the seven greatest storms of record in the general region, as obtained from Corps of Engineer Reports (Refs. 22-23) or Studies from which it can be seen that the 3 greatest storms of record for 2800 square miles were those of August 6-9, 1940 in southwestern Louisiana; June 27-July 1, 1899 centered at Hearne, Texas; and September 6-10, 1921 centered at Taylor, Texas. These latter two have been carefully studied by the Corps of Engineers and they had (1944) under study the Louisiana storm, for which they furnished a print of the isohyetal map of the total storm and a computation sheet giving the maximum average depth of rainfall for various areas and durations as developed from this storm. However, the Consulting Engineers found it necessary to make their own complete analysis of this storm, using information made available by the Hydrologic Unit of the Regional Office of the Weather Bureau at Fort Worth, Texas, including "Special Supplement No. 1 Covering the Storm of August 6-10, 1940 in Louisiana and Texas."

All of the great storms originated over the Gulf and each was considered for transposition to the San Jacinto basin. The three greatest storms of record were all believed transposable to the San Jacinto with the possible exception of the Taylor, Texas, storm of September 1921 in which the orographic effect of the rising topography was present. In the interest of a conservative appraisal of possibilities, however, it was assumed that the 1921 Taylor storm, with no reduction of either rate or amount of rainfall, could have occurred over the San Jacinto basin. Since the August 1940 storm proved to be the one which would have produced the greatest peak flow at the dam site, it only will be discussed in detail here. The description of the storm is from the Special Supplement of the Fort Worth Weather Bureau office.

This storm, a tropical hurricane, moved inland over the extreme western Louisiana coast on


12 HR GROSS RAINFALL = 0.70"
ESTIMATED LOSS = 0.70"
EXCESS RAINFALL = 0.00"
TRANSPOSED WATERSHED
Non-Recording Rain Gage
Recording Rain Gage
ELTON
EUNICE
GRAND COTEAU
PENTON
IOWA
ROANOKE
JENNINGS
CROWLEY
LAFAYETTE
SCOTT
BROUSSARD
MAURICE
MILTON
CADE
ATCHAPALAYA
PLAQUEMINE
CARVILLE
WHITE CASTLE
LAKE ARTHUR
GUEYDAN
KAPLAN
ABBEVILLE
ERATH
DUBOIN
JEANERETTE
CHARENTON
BELLEROSE
PAINCOURTVILLE
NAPOLEONVILLE
ESTHER
FRANKLIN
GRAND CHENIER
PATTERSON
CENTERVILLE
MORGAN CITY
PECAN ISLAND
HOUSTON, TEXAS
SAN JACINTO RIVER HYDROLOGIC STUDIES
ISOHYETAL MAP OF LOUISIANA STORM OF AUG. 6-9, 1940
FOR 12 HOURS ENDING 8 A.M. AUG. 6, 1940
GRAPHIC SCALE
40 MILES


12 HOUR GROSS RAINFALL = 6.40"
ESTIMATED LOSSES = 0.68"
EXCESS RAINFALL = 5.72"
TRANSPOSED WATERSHED
Non-Recording Rain Gage
Recording Rain Gage
ELTON
EUNICE
GRAND COTEAU
PENTON
IOWA
ROANOKE
JENNINGS
CROWLEY
LAFAYETTE
SCOTT
BROUSSARD
MAURICE
MILTON
CADE
ATCHAPALAYA
PLAQUEMINE
CARVILLE
WHITE CASTLE
LAKE ARTHUR
GUEYDAN
KAPLAN
ABBEVILLE
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CENTERVILLE
MORGAN CITY
PECAN ISLAND
HOUSTON, TEXAS
SAN JACINTO RIVER HYDROLOGIC STUDIES
ISOHYETAL MAP OF LOUISIANA STORM OF AUG. 6-9, 1940
FOR 12 HOURS ENDING 8 P.M. AUG. 9, 1940
GRAPHIC SCALE
40 MILES

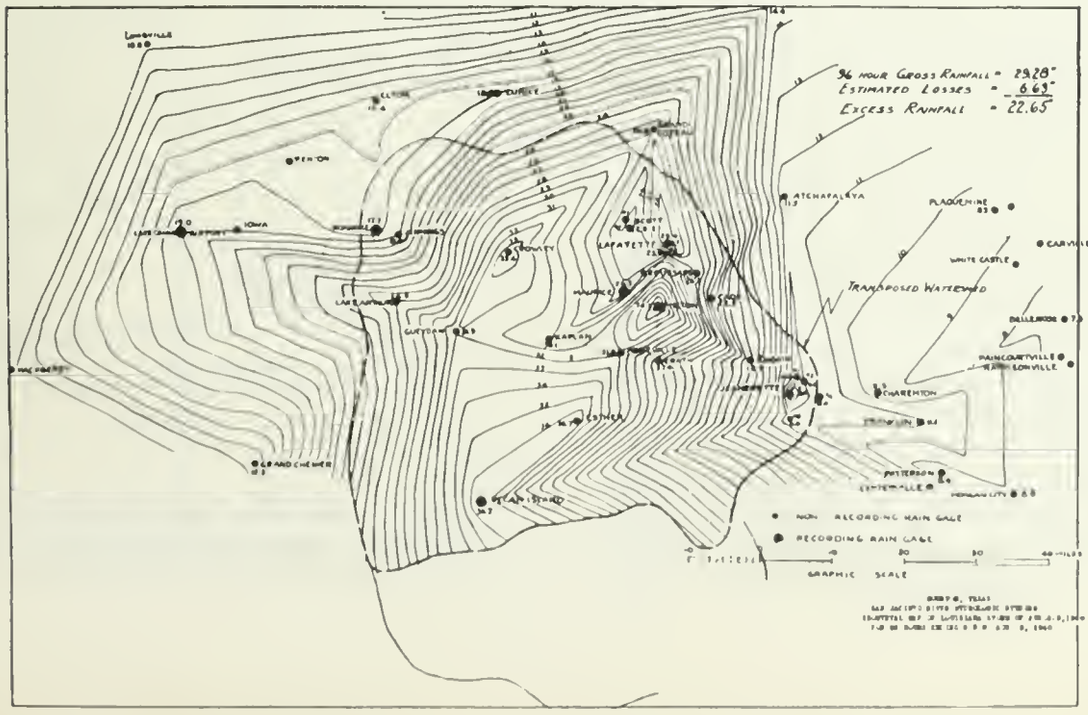
96 HOUR GROSS RAINFALL = 29.28"
ESTIMATED LOSSES = 6.63"
EXCESS RAINFALL = 22.65"
TRANSPOSED WATERSHED
ELTON
EUNICE
PENTON
IOWA
ROANOKE
JENNINGS
CROWLEY
LAFAYETTE
SCOTT
BROUSSARD
MAURICE
MILTON
CADE
ATCHAPALAYA
PLAQUEMINE
CARVILLE
WHITE CASTLE
LAKE ARTHUR
GUEYDAN
KAPLAN
ABBEVILLE
ERATH
DUBOIN
JEANERETTE
CHARENTON
BELLEROSE
PAINCOURTVILLE
NAPOLEONVILLE
ESTHER
FRANKLIN
GRAND CHENIER
PATTERSON
CENTERVILLE
MORGAN CITY
PECAN ISLAND
NON RECORDING RAIN GAGE
RECORDING RAIN GAGE
GRAPHIC SCALE

FIG. 50.

August 7, 1940. Moving northwestward, the storm passed about eight miles south of Port Arthur, Texas, and then curved gradually to the north and northeastward. The path was a short distance west of Beaumont, thence through Hardin, Tyler, eastern Angelina, eastern Nacogdoches, northwestern Shelby, Panola and extreme eastern Harrison Counties, all in Texas, thence into Louisiana a short distance northwest of Shreveport. After striking Port Arthur, the disturbance diminished rapidly in intensity except for torrential rains in western Louisiana and the extreme eastern part of Texas. At Port Arthur the highest wind recorded was 82 miles per hour from the northeast. The approximate center of precipitation for the storm lies about 50 miles south and 190 miles east of the center of the San Jacinto Basin above the dam site. There are no topographic features of significance which should be given consideration in assuming a transposition of this storm to a position over the San Jacinto Basin. Other tropical hurricanes, notably the August 1915 storm, passed inland farther to the west and it is considered quite valid to assume that a storm similar to the Louisiana storm of 1940 could occur, centered over the San Jacinto Basin. Accordingly, on Fig. 50 have been endorsed the outlines of the watershed of the San Jacinto Basin with no rotation of the directional axes. From the data in the Special Supplement supplied by the Fort Worth Regional Office of the Weather Bureau, mass curves were drawn for each of the rainfall stations in the area of heaviest precipitation. From these mass curves isohyetals were drawn for each 12 hours of the storm. Three of these are shown on Fig. 50. Rainfall quantities within the transposed watershed line were accumulated and the resulting 12-hour amounts were endorsed on Fig. 51.

In Ref. 22 it was noted that the storm of March 14-15, 1929, found the cold, modified polar air a short distance below the Gulf Coast. Other runoff and isohyetal charts of that report established the possibility of a cold air mass forcing the warm moisture-laden Gulf air to rise and precipitate its moisture anywhere between the coast and farther inland than even the Hearne, Texas, 1899 storm.

It will also be noted that on Fig. 51 there have been shown the abstractions or "losses" for each of the 12-hour periods of rainfall. These are based upon the assumption of wet antecedent conditions with infiltration values assumed varying as shown. These infiltration rates were chosen from the data developed for sandy soils at the Tyler, Texas, Soil and Water Conservation Experiment Station of the Soil Conservation Service. The excess precipitation for each 12-hour period was applied to the adopted unit graph to obtain the probable hydrograph resulting from a storm similar to the August 5-9, 1940, Louisiana tropical hurricane if it had been centered over the San Jacinto watershed. Attention is directed to the mass rainfall, "losses" and runoff which were respectively 29.3, 6.6 and 22.7 inches over the entire 2840 square miles.

Yield Studies for San Jacinto Water Supply Reservoir. The probable dependable stream flow at the dam site is one of the most important determinations required in the engineering of a water supply impoundment. In most instances, as was true for the San Jacinto River impoundment for supplemental water supply for Houston, Texas, the rainfall records are much longer than the available stream-flow data and the initial step is an examination of the precipitation data to determine and compare the periods of deficient rainfall, and if possible select a period which might reasonably result in the critical minimum runoff for the watershed.

For this specific problem, there was next developed a correlation between the very short stream-flow record at Huffman (which fortunately is almost at the chosen dam site) and the somewhat longer record at Humble, so this latter could be transferred to Huffman. From the recorded stream flow at Humble and the rainfall data for the same years, it was possible to develop a relationship between rainfall and runoff which might thereafter be applied to the rainfall of the selected dry period and thereby determine the probable runoff during the selected period of the probable lowest yield. With this computed runoff, a mass curve of inflow into the proposed reservoir can be prepared and studies of dependable yield of reservoirs of various capacities can be based upon it.

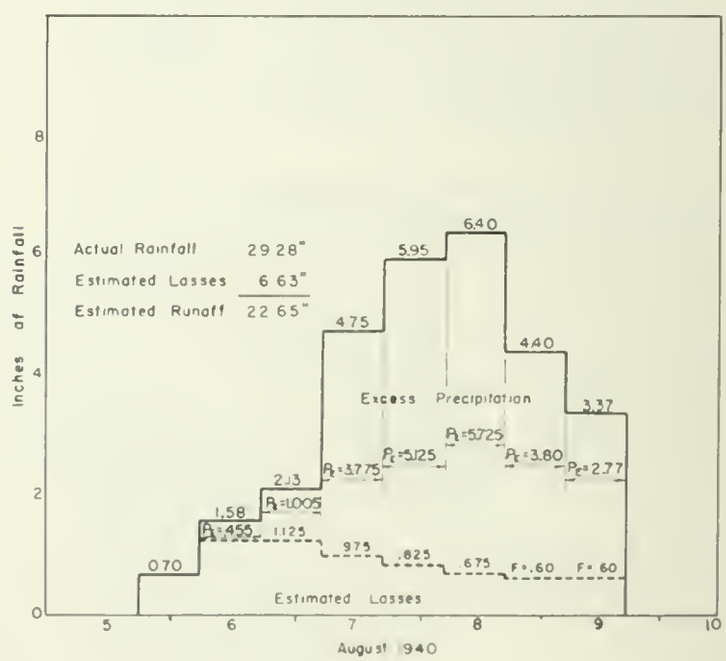

FIG. 51.—SAN JACINTO RIVER HYDROLOGIC STUDIES, EXCESS PRECIPITATION, TRANSPOSED STORM OF AUGUST 8-9, 1940.

Table III

Seasonal Rainfall during Drouth Periods
1889 - 1943

Period**	Minimum Weighted Rainfall in Inches*					
	One Season	Two Seasons	Three Seasons	Four Seasons	Five Seasons	Six Seasons
Nov. '92-Oct. '99	Nov. '94-Mar. '95 12.24	Apr. '94-Mar. '95 34.03	Nov. '92-Mar. '94 53.31	Apr. '93-Mar. '95 71.70	Nov. '92-Mar. '95 87.34	Nov. '92-Oct. '95 115.38
Apr. '00-Oct. '04	Nov. '03-Mar. '04 7.73	Apr. '01-Mar. '02 26.69	Nov. '00-Mar. '02 44.95	Nov. '00-Oct. '02 78.78	Nov. '01-Mar. '04 103.42	Apr. '01-Mar. '04 118.74
Apr. '08-Mar. '13	Nov. '08-Mar. '09 7.89	Nov. '08-Oct. '09 30.69	Nov. '06-Mar. '10 43.44	Nov. '08-Oct. '10 68.29	Nov. '08-Mar. '11 81.26	Nov. '08-Oct. '11 108.73
Apr. '15-Oct. '19	Nov. '17-Mar. '18 8.23	Apr. '17-Mar. '18 20.67	Nov. '16-Mar. '18 30.73	Nov. '16-Oct. '18 54.19	Nov. '15-Mar. '18 68.29	Nov. '15-Oct. '18 91.75
Apr. '24-Mar. '27	Nov. '24-Mar. '25 9.43	Apr. '24-Mar. '25 33.41	Apr. '24-Oct. '25 58.16	Apr. '24-Mar. '26 85.18	Apr. '24-Oct. '26 113.52	Apr. '24-Mar. '27 130.97
Nov. '30-Mar. '36	Apr. '32-Oct. '32 15.38	Apr. '32-Mar. '33 34.92	Apr. '32-Oct. '33 53.72	Apr. '32-Mar. '34 75.71	Apr. '32-Oct. '34 93.67	Apr. '32-Mar. '35 117.12
Apr. '37-Oct. '40	Nov. '39-Mar. '40 13.44	Apr. '39-Mar. '40 33.86	Nov. '38-Mar. '40 54.91	Apr. '38-Mar. '40 79.20	Nov. '37-Mar. '40 96.97	Apr. '37-Mar. '40 117.14

*Weighted seasonal rainfall on watershed above dam site.
**Period includes at least one wet season at beginning and at least one wet season at end.

The monthly rainfall totals for all of the stations in and adjacent to the San Jacinto Basin were carefully weighted by the Thiessen method to determine the equivalent uniform rainfall for both the entire basin above Huffman and for the West Fork above Humble. From these weighted monthly rainfalls for April 1889 to October 1943, there were prepared mass diagrams, monthly bar diagrams with the deficiencies below the monthly means shaded, and Fig. 52 which shows the 3-year moving average annual weighted rainfall for the basin. The most critical period of rainfall deficiency was shown by these plottings to be the period from 1916 to 1918.

Further study revealed that the runoff for most years is divided into two periods of distinctly different characters, with the tendency for seasonal changes to occur on the average about April 1st and November 1st. Accordingly, there were selected the five-month season of November to March inclusive, and the seven-month season of April to October inclusive. It was also recognized that while there is not the usual distinct difference between growing and dormant seasons in the San Jacinto watershed, nevertheless, since rainfall is fairly uniformly distributed throughout the year, it must be concluded that the sharp change in runoff character is primarily related to a distinct change in water "losses" and therefore to a similar distinct change in mean temperature conditions.

A study of monthly and seasonal mean temperatures gave an average of 53.8° F. for the five-month dormant season, and 76.6° F. for the seven-month growing season. This brings out sharply the distinctive differences which must exist with respect to evaporation, transpiration and water losses. And this was corroborated by the seasonal correlations of rainfall and runoff, which indicated mean seasonal rainfalls with identical monthly averages for each of the two seasons, but 0.73 inch monthly average runoff during the mean five-month dormant season as compared to 0.40 inch during the seven-month growing season.

In Table III there are indicated the seasonal rainfalls for 1 to 6 consecutive seasons for the seven drouth periods of the rainfall record from 1889 to 1943. It will be noted that with the exception of the single season, for which November 1903 to March 1904 had the lowest rainfall, the minimums for 2, 3, 4, 5 and 6 consecutive seasons all occurred in the general period April 1915 to October 1919. It should be noted here that in a few instances runoff at the beginning of a season (for which stream-flow records were available) was due to rainfall at the end of the preceding season. For such cases adjustments were made to include the related rainfall and runoff in the same seasonal totals.

The yield studies proceeded to develop the relationship between seasonal rainfall and actual runoff for each of the two seasons and this was then

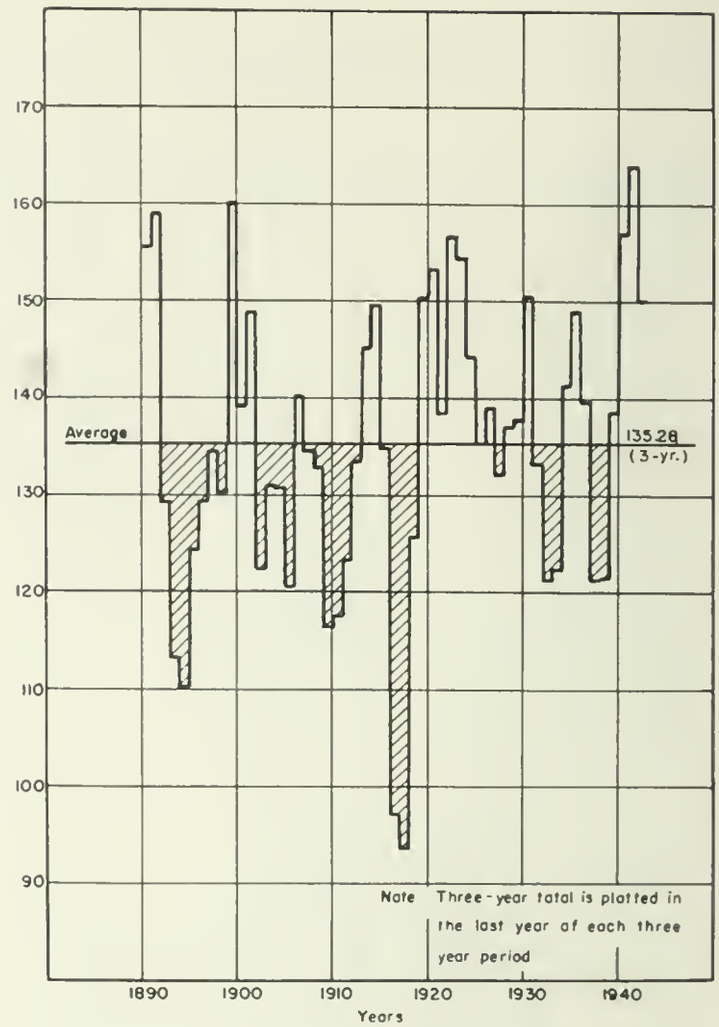

FIG. 52.—SAN JACINTO RIVER HYDROLOGIC STUDIES, THREE-YEAR TOTAL RAINFALL, 1889-1943. WATERSHED ABOVE DAM SITE.

applied to the seasonal values in the selected drouth period of 1915-18 to determine the minimum probable stream flow for this period. Similar determinations were made for three consecutive seasons, embodying two groups: April-October, November-March, April-October; and a second group with a growing season between two dormant ones. Extensive probability studies were made primarily to secure a characterization of the selected drouth period in terms of frequency of occurrence; these also were useful as a guide to drawing maximum loss curves.

The probability studies indicated that while the 1915-19 drouth occurred within a period of 50 years, its probability of recurrence is actually on the order of once in 200 years or greater. Even if there is recognition of the questionable validity of using probability studies of this kind in their extreme range, it still appears safe to consider this drouth period at least on the order of a 100-year frequency.

The finally recommended mass curve of res-

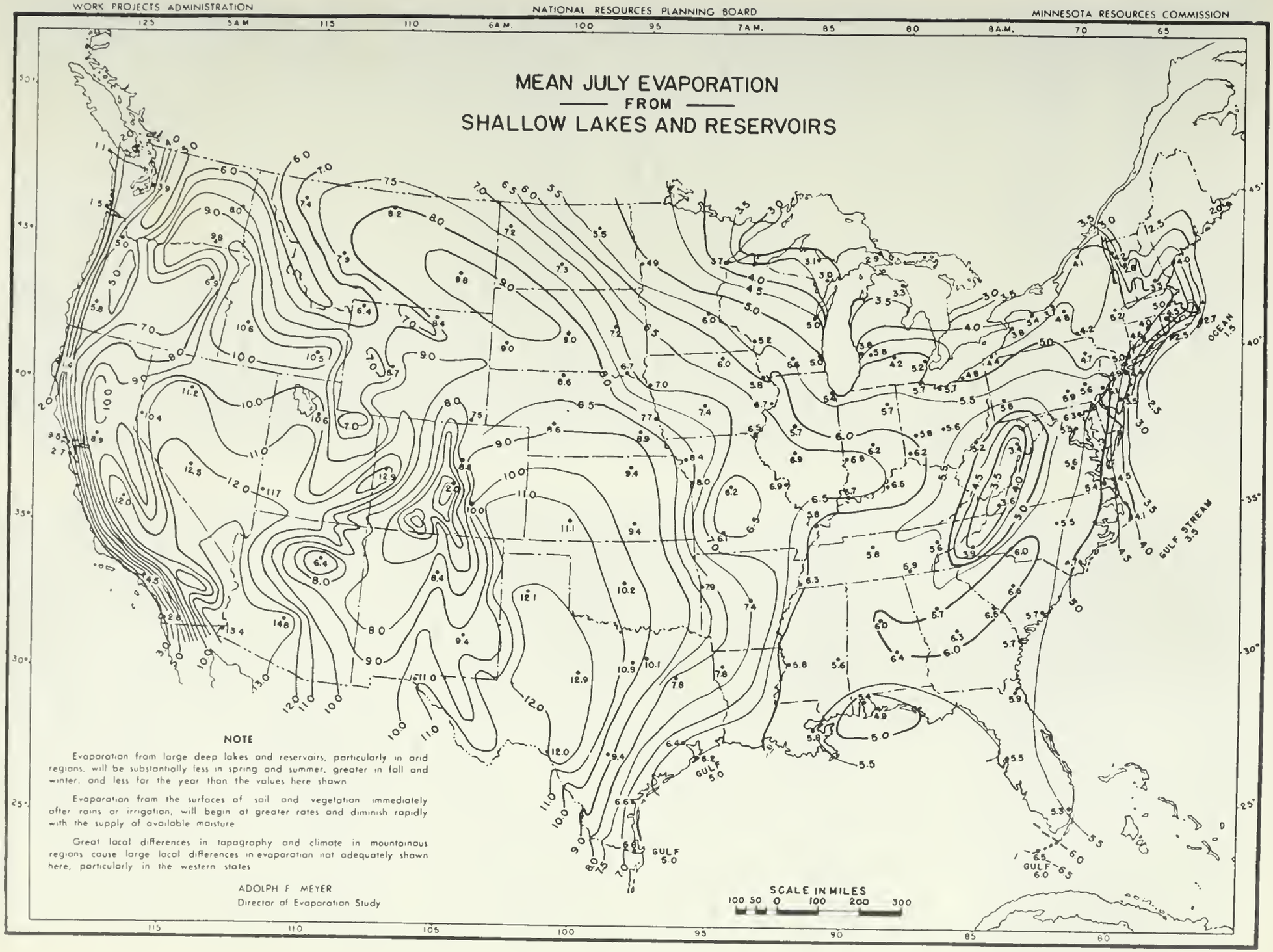
WORK PROJECTS ADMINISTRATION
NATIONAL RESOURCES PLANNING BOARD
MINNESOTA RESOURCES COMMISSION
MEAN JULY EVAPORATION
FROM
SHALLOW LAKES AND RESERVOIRS
NOTE
Evaporation from large deep lakes and reservoirs, particularly in arid regions, will be substantially less in spring and summer, greater in fall and winter, and less for the year than the values here shown
Evaporation from the surfaces of soil and vegetation immediately after rains or irrigation, will begin at greater rates and diminish rapidly with the supply of available moisture
Great local differences in topography and climate in mountainous regions cause large local differences in evaporation not adequately shown here, particularly in the western states
ADOLPH F MEYER
Director of Evaporation Study
SCALE IN MILES
100 50 0 100 200 300
GULF STREAM 3.5
OCEAN 1.5
GULF 5.0
GULF 6.0

FIG. 53.

ervoir inflow incorporated the use of minimum three-seasonal runoff as obtained by subtracting the maximum "losses" from the actual three-seasonal rainfall for the critical period. For the 18 months from April 1, 1917 to October 31, 1918, the computed runoff was 1.65 inches.

In developing a realistic demand curve for a water supply impoundment, it is necessary to evaluate, among other factors, the probable evaporation. There has been prepared a report (Ref. 25) on evaporation from lakes and reservoirs based upon 50 years of Weather Bureau records. The results are in admirably clear and practical form, including charts giving mean evaporation for each month of the year. This presents the practising engineer with evaporation data comparable to the point rainfall data as presented by Yarnell (Ref. 6). A sample reproduction of one of the charts is included here as Fig. 53 and attention is directed to the cautions and limitations in the "Note."

The engineer concerned with yield studies is referred to Ref. 26, p. 579ff. for an excellent brief discussion of droughts, including a good bibliography of the literature up to then (1942).

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DISCUSSION

GEORGE S. BENTON.*—I have followed with interest Mr. Jens' discussion of "Engineering Meteorology." It is a clear and concise presentation of current techniques employed by hydrologists in the analysis of precipitation data. The report raises several questions in my mind, however. Can the relation between the sciences of meteorology and hydrology be described adequately in terms of techniques of analysis of precipitation records? More basically, is the conventional attitude of the hydrologist, toward meteorology in general and hydrometeorology in particular, sufficiently broad to enable him to cope with problems which have been raised in the past few years—and problems which will be raised in ever increasing number during the coming decade?

As a meteorologist, my own answers to these questions are in the negative. It is my firm conviction that the relation between the fields of hydrology and meteorology is going through an uneasy period of readjustment. New concepts and new attitudes are being evolved by hydrologists which bear little resemblance to the concepts and attitudes which they are beginning to replace. This process will be accelerated in the future, as the science of hydrometeorology develops.

A hint of things to come may be found in the session on Radar-Weather which is being held concurrently with this Hydrology conference. The meteorologist is developing powerful new tools which are of great potential value to hydrology. Yet these new tools can be of maximum utility only if the hydrologist has the interest and the know-how to use them to best advantage. I do not mean by this that the practicing hydrologist must become an expert in the design or even the physical operation of electronic equipment such as radar. He must, however, develop an attitude towards and a knowledge of meteorology which he lacks today.

The majority of hydrologists in the past have been content to use precipitation records as a set of basic statistical data, and have evidenced only a superficial interest in the physical processes on a large and small scale which have given rise to the distribution in space and time of the quantity which they are studying. Hydrologists may point out that sections of every text in hydrology include a chapter on the cause of precipitation; that virtually every university course in hydrology starts with a few lectures on meteorology; and that occasional technical articles on meteorological subjects, authored by hydrologists, have appeared in engineering journals. Yet with few exceptions, these facts are excellent examples of the inadequacy, rather than the adequacy, of the training and experience of the average hydrologist in subjects meteorological. Texts in hydrology usually contain some such statement as the one appearing in a recent and very popular reference (1)*: "A discussion in anything like adequate detail, of the causes which produce precipitation and which influence its distribution, its total amount, and its variation would be too extended for our purposes."

This is typical of what I might call the conventional view of hydrologists towards meteorology: hydrologists need not "for their purposes" be concerned with the physics of precipitation or the movement of water vapor in the atmosphere; it is sufficient merely to obtain and use the precipitation records which are available from a variety of sources.

This attitude I have always found understanding but disturbing. Why should the hydrologist lose interest in water when it rises from the surface of the earth in vapor form through the processes of evapo-transpiration? Why should he be interested in precipitation only from the moment that it reaches the surface of the earth as rain or snow? A competent hydrologist would not think of analyzing the records of well yield from a group of ground water wells without relying heavily on his knowledge of the physics of the occurrence and movement of water below the surface of the earth. Yet this restraint vanishes when precipitation data are presented to him. In this instance he often assumes that a superficial toward the physics background is a sufficient prerequisite to his analysis. This fact can be further emphasized by a simple comparison. The concept of condensation and sublimation nuclei in the atmosphere is as fundamental to an understanding of the formation of precipitation as the concept of permeability is to an understanding of the movement of fluids through a porous system. However, an examination of the various texts in hydrology which have been published in this country during the last ten years shows that in only one instance was the existence of condensation nuclei or sublimation nuclei in the atmosphere even mentioned—and even in this exceptional case, the discussion was restricted to a few sentences.

I think it is important to examine why this curious dichotomy exists in the attitude of the hydrologist. Why has physical meteorology been placed in the role of a second-class relation? The answer, I believe, lies in three facts. First, a knowledge

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*See BIBLIOGRAPHY at end of this discussion for all references.

of the physics of precipitation has not in the past been of as vital practical importance to the hydrologist as the understanding of certain other physical processes. The reason for this lies in the fact that the control of weather phenomena has been considered impossible, whereas the possibility of regulation of well yield, for example, is obvious. Where the possibility of control exists, the knowledge of how various parameters affect the phenomenon under examination becomes critical. A second reason why the physics of precipitation has been neglected by hydrologists is based upon the complexity of the problems involved in meteorology. The atmosphere is a compressible, heterogeneous fluid in three dimensional unsteady state motion on a rotating earth. There are external sources of radiational energy and internal sources for the release of latent energy. From the viewpoint of fluid mechanics and thermodynamics, the dynamics of the atmosphere is almost unbelievably complex. The hydrologist is already burdened by a vast field which includes many complex physical phenomena. He can hardly be berated, therefore, for having abandoned the detailed study of the physics of the atmosphere to the professional meteorologist. However, once this principle had been enunciated, the hydrologist tended to dismiss from his mind all problems of atmospheric physics. He began to build his own edifice of hydrologic science independent of, rather than closely connected to, the growing structure of meteorological knowledge. A third reason for the attitude of the hydrologist towards precipitation data arises from engineering necessity. Hydrologists were confronted with problems of analysis of precipitation data long before the science of meteorology had developed to a point where the study of the physics of the atmosphere could be of significant help to them. Hydrologists had no alternative, therefore, but to make the best of the information at their disposal. However, it gradually became an occupational assumption that a careful study of meteorological theory was superfluous to the hydrologist's needs. This attitude was never entirely justified. It is even less justified at present, for during the past few years the science of meteorology has been developing at an astounding rate.

It has been inevitable, in view of the above facts, that the hydrologist should encounter increasing difficulty in his use of meteorological information. Certain errors of thinking appearing at an early date are familiar to all of you. The inadequacies of the original station-year techniques of analyzing precipitation data were based in large measure on a lack of appreciation of the areal interdependence of precipitation records between stations (2). The erroneous concept of the source of water for precipitation, i.e., the idea of land-derived evapotranspiration serving as the primary source of rainfall, was a direct result of inadequate understanding of the mobility of the atmosphere and the lack of a close relationship between precipitable water content of air and amount of precipitation (3, 4). Other such difficulties could be enumerated. All are examples of an inadequacy in the hydrologist's training in meteorology; however, none are of such vital importance to the field of hydrology that they served to drive home to the practicing hydrologist his need for a physical approach to meteorology problems.

In recent years, however, new questions have arisen which are forcing a gradual revision of the conventional attitudes of the hydrologist towards meteorology. One of these—the uses and potentialities of radar—has already been mentioned. Among the most dramatic of the others are questions raised by recent experimentation in the artificial nucleation of convective clouds. This is not the proper place for a discussion of the merits and weaknesses of the methods which have been suggested for inducing precipitation. It is important to point out, however, that meteorologists themselves are divided in their evaluation of the efficacy of these techniques. The great majority believe that artificial nucleation as practiced at present is not economically, or hydrologically, important. They feel that many of the commercial ventures for increasing rainfall are being conducted by persons more interested in financial return than in alleviating water shortages or promoting scientific knowledge of cloud physics. A minority of meteorologists, however, insist that artificial nucleation is already of great economic significance and that the majority are reactionaries who are unable to see and appreciate the scope and import of daring and new ideas.

In the resulting confusion of charge and countercharge, the hydrologist has been placed in a precarious position. In many instances, it is he who must decide whether an organization's time and financial resources are to be placed at the disposal of the meteorologist sponsoring a program of artificial nucleation. Yet there is little in the hydrologist's background to prepare him to make such a decision. In most similar situations, normal business procedure would counsel a conservative investment of funds, with an expansion of activities to be contingent upon successful accomplishment. In this case, however, the success or failure of an artificial nucleation program is far from clear. In fact, the "success" or "failure" of a program invariably depends far more on the meteorologist preparing the analysis than upon the data under examination. Any meteorologist worthy, or unworthy, of his calling can prepare an impressive array of statistics in support of either or both points of view. The time-honored methods of hydrologic analysis of meteorological data are of little help under such circumstances.

What is the hydrologist to do today?

The hydrologist obviously cannot make himself over into a professional meteorologist in order to meet the new meteorological problems with which he is confronted today. However, two things are happening under the impact of these stimuli. First,

the hydrologist is realizing the importance of broadening his background in physical meteorology, so that he can more adequately evaluate new problems of development in the border area between hydrology and meteorology. This requires and must accompany a more basic approach to fluid mechanics and its applications. Second, a small but growing group of professional meteorologists are devoting themselves to border problems between the two fields. The science of hydrometeorology has been born within the last decade. The hydrometeorologist of today must be thoroughly trained in physical meteorology, in cloud physics, in the causes of precipitation, and in the morphology of storms in space and time, three directional as well as surface; however, he must also be thoroughly familiar with the uses of precipitation data and with the various practical problems continually being faced by the hydrologist.

In the Civil Engineering Department of the Johns Hopkins University a program is now under way which seeks to encourage these developments. Graduate study in meteorology, in hydrometeorology, and in hydrology is presented in an organized manner, as parts of a unified whole. Candidates for advanced degrees in hydrology must, as a matter of course, study basic fluid mechanics and its application to the problems of meteorology and hydrometeorology. On the other hand, meteorologists are acquainted at an early stage in their training with the problems encountered in hydrology. The results of such an approach have thus far been stimulating and encouraging.

Apart from the considerations discussed above, it is important to point out that additional benefits can accrue and are accruing to the field of hydrology by the growth of the science of hydrometeorology. There is no reason why the field of hydrology cannot be extended as a matter of course to include the atmosphere above the continents. For example, the concept of the hydrologic cycle as it is conventionally presented by hydrologists is weak and unnecessarily restrictive. It would seem clear that the transport of water vapor across the boundaries of a watershed by the atmosphere is every bit as essential a part of the hydrologic cycle as water losses across this boundary by deep seepage. This broadened point of view towards the hydrologic cycle has significant implications with regard to the hydrologic balance of an area. For example, as has been noted elsewhere, this approach may be of basic importance in the computation of evapo-transpiration losses from large areas for periods of time of the order of a week or a month (5).

The hydrometeorologist is learning a great deal about the movement of water vapor by the atmosphere. One of our hosts, Mr. Stout of the Water Survey Division, has done some valuable work in this respect. An ambitious program is also well under way at the Johns Hopkins University, under the sponsorship of the United States Air Force. There, I and my collaborators have been compiling data on water transfer by the atmosphere for the entire North American continent and the surrounding oceans. Data extracted from weather charts are being entered on punch cards. When the first stage of the program is completed, water transport data for the entire calendar year of 1949 will be available at all levels of the atmosphere. This information will be used, in part, to clarify our concept of the role of the atmosphere in the hydrologic cycle. It will also be used to analyze the water budget of storms in an attempt to clarify their physical nature as hydrometeorological entities.

The coming years will see a revolution in the hydrologist's attitudes towards meteorology. The hydrometeorologist of today will, I am certain, prove to the hydrologist, if such proof is necessary, that a physical approach to problems of precipitation will yield advances of extreme value and significance in the field of hydrology. These changing relationships between hydrology and meteorology are, to me, of unique importance in evaluating Engineering Meteorology today.

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IVAN E. HOUK.*—Mr. Jens has presented an excellent paper on the "Engineering Understanding and Use of Rainfall Data." His presentation of the subject is well developed, adequately illustrated, and supplied with such pertinent references as may be needed in order to secure further explanations.

The use of rainfall data is the phase of engineering meteorology that is commonly involved in the work of civil engineers, especially those engaged on hydrological problems. There are several other phases of engineering meteorology, some of which are mentioned by Mr. Jens at the beginning of his paper. Because of the complexity of the general subject, a few additional remarks may be justified.

In the first place, what is engineering meteorology? Does it need a definition? Probably the terms engineering and meteorology are so well understood that the combined term—engineering meteorology—is in itself fully explanatory.

The Weather Glossary, published by the United States Weather Bureau in 1946, lists ten subdivisions of meteorology, but does not include engineering meteorology. It includes hydrometeorology, which constitutes one phase of engineering meteorology.

Aeronautical meteorology has been defined as "The branch of Meteorology concerned with weather insofar as it affects aviation." Possibly engineering meteorology might be defined as the branch of meteorology concerned with weather insofar as it affects engineering.

Scientists dealing with weather phenomena usually think of weather as atmospheric conditions during a relatively short period of time. They think of climate as weather conditions during a comparatively long period of time, including maximum and minimum occurrences as well as average data. They think of meteorology as the science that deals with atmospheric phenomena and the basic laws that produce and control such phenomena.

Actually, all three—weather, climate, and meteorology—are so interrelated that definite boundaries are difficult to establish. Most people, including many engineers, use the three terms more or less synonymously.

Elements of meteorology that determine the characteristics of the atmosphere are:

Temperature	Storms
Radiation	Precipitation
Sunshine	Evaporation
Cloudiness	Barometric pressure
Fog	Composition of air
Humidity	Electrical phenomena, and
Wind	Optical phenomena.

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One or several of these elements, with the possible exception of the last, may be involved in some type of engineering work. Many examples could be mentioned, but a few will suffice for this discussion.

Mechanical engineers take into account weather conditions in planning the installation of heating, refrigeration, ventilating, and air-conditioning systems.

Electrical engineers make allowances for weather conditions in planning hydroelectric plants, especially where subfreezing temperatures may cause frazil and anchor ice troubles at power intakes. They also consider weather in planning transmission lines through regions where ice and sleet storms may cause severe damages.

Structural engineers consider temperature changes, wind forces, and accumulated snow loads in designing bridges, buildings, and other structures.

Civil engineers consider weather elements in planning streets, roads, bridges, sewers, drainage systems, dams, reservoirs, river improvements, airports, and other items of construction that may be affected by heavy snowfalls, storm rainfall, and flood runoff. In such work, atmospheric elements of prime importance are temperature and precipitation. In planning storage projects, rates of evaporation and total evaporation losses constitute pertinent considerations, especially in western United States.

The particular phase of engineering meteorology which Mr. Jens has taken up in detail is commonly referred to as hydrometeorology. Hydrometeorology may be defined as the branch of science that deals with atmospheric moisture, precipitation, disposal of precipitation, and effects produced by precipitation, particularly those caused by flood runoff.

This subdivision of meteorology, or of engineering meteorology, has been receiving more and more attention since the disastrous floods of March 1913 in the Ohio River valley and the subsequent intensive studies of storm rainfall conducted by The Miami Conservancy District.

In 1937, the Hydrometeorological Section of the United States Weather Bureau was organized to study storm rainfall, resultant flood runoff, and the various physical and atmospheric conditions that produce and control such phenomena. This Section, under the able directorship of the late Merrill Bernard and in cooperation with certain other agencies of the Federal Government, has been investigating and developing methods for evaluating the most intense flood-producing meteorological conditions that can occur in particular drainage basins. This question has always been one of the most troublesome and elusive problems that engineers have had to solve. Thus far, the Section

has completed more than twenty reports on its investigations.

Engineers have often criticized meteorological equipment and methods of observing and recording the weather of the country. They have found fault with rain gages, snow gages, anemometers, evaporation pans, and other scientific instruments used in measuring atmospheric conditions. They have objected to locations of observation stations, methods of installing measuring apparatus, procedures followed in taking readings, and other factors involved in determining and recording meteorological phenomena.

Constructive criticism is always desirable. Engineers, scientists, and others employed on important work should always strive for the best possible results. Many of the faults found with meteorological observations probably were well founded. Nevertheless, I believe that past criticisms of our weather service have sometimes been overdone.

One fault often found with weather data in the past concerned the paucity of records. Fortunately, this fault has been corrected to a large degree during recent years. One important factor in the extension and enlargement of our weather service has been the development of aviation and the increasing needs for reliable meteorological information in providing safe airplane transportation. Many first-order weather stations have been established and are now being maintained at airports throughout the country.

Meteorological phenomena often are so variable and erratic that the items of information needed by engineers probably never can be determined with 100 per cent efficiency. For instance, the cloudbursts that are experienced along the slopes of the Wasatch Mountains, along the Front Range of the Rockies, and to some extent in other parts of the country, are of such a local nature that we can hardly hope to have rain gages installed at all places where they occur.

Mr. Jens, in his discussion of intense rainfall and point rainfall data, has referred to The Miami Conservancy District Technical Reports, the Yarnell report, and certain other sources of information. Perhaps the report by Shands and Ammerman on "Maximum Recorded United States Point Rainfall . . ." might also be mentioned. This report, published as Weather Bureau Technical Paper No. 2, 1947, gives maximum rainfalls as recorded at 207 first-order stations during periods of 5, 10, 15, 30, and 60 minutes, also during periods of 2, 3, 6, 12, and 24 hours. It supplements and amplifies the Yarnell report and includes data recorded up to the end of 1945.

In connection with considerations of intense rainfall, it might be interesting to mention the sharply defined edges of rainstorms that are sometimes reported. Until recently, I have always taken such reports with one or two proverbial grains of salt. However, one afternoon last August, I looked out the back windows of my home and saw raindrops falling thick and fast. Upon facing about and looking through the front windows, I saw absolutely no rain at all.

Changes in air temperature, too, are sometimes abrupt and large, especially in regions where chinook winds occur. At Spearfish, South Dakota, a rise of 49 degrees in 2 minutes was observed in January 1943. Such extreme changes in temperature and rainfall, although probably not of great importance in most engineering work, do illustrate the vagaries of weather and the difficulties involved in accurately observing and recording meteorological data.

Engineers engaged on special projects where meteorological phenomena constitute important considerations may have to adjust existing weather records to meet their particular needs. For instance, they may have to adjust rainfalls measured at different stations to the same time period, so that isohyetal charts can be prepared for the area under study; or they may have to correct wind velocities measured on the tops of high office buildings to ground values, for use in evaporation studies. Other adjustments needed for special purposes might be mentioned.

In the case of unusually large and important projects, engineers may have to install additional stations, in order to secure more detailed data on atmospheric elements. This is often done by federal, state, and municipal agencies; also by power companies and other organizations engaged in the conservation, control, and use of water resources. The Bureau of Reclamation, Corps of Engineers, and Tennessee Valley Authority can be cited as examples.

The fact that engineers sometimes have to adjust meteorological records, or install stations for securing additional data, does not constitute a reflection on our national weather service. Weather data needed by engineers often differ from weather data needed by persons engaged in other types of work. Our national agencies must supply, insofar as possible, the data needed by persons engaged in all types of activities. To me, it seems remarkable that existing weather records throughout the country are as comprehensive and as well suited to the requirements of engineers as they now are.

Mr. Jens, in his well-developed presentation, has shown how rainfall records and related meteorological data can be processed and adapted to the solution of engineering problems involving hydrological considerations. Other engineers might use slightly different procedures in correlating and analyzing the data. However, his paper contains but few, if any, points that could be picked out as a basis for extended criticism or argument.

PHILLIP LIGHT.* — The author presents a valuable summary of the important tools and procedures for handling rainfall data in engineering design. The formulae and charts described in the paper are derived from statistical treatment of rainfall data, but despite some limitations have had very useful applications. The science of meteorology has advanced rapidly in recent years due mainly to an increased knowledge of the upper air, but there is a large gap between development of theory and application to practical engineering problems. The results of many meteorological investigations are only qualitative, but it is likely that a better understanding of atmospheric phenomena will lead ultimately to refinements of empirical relations.

The Yarnell and Miami Conservancy District charts of rainfall intensity vs. duration and frequency date back to 1933 and 1934. Since that time there has been a progressive increase in the number of automatic rain gages, and a large accumulation of valuable records. The frequency calculations are subject to a sampling error, which depends on the length of record and number of rainfall stations. It is, therefore, important to re-examine these charts, particularly those for short durations and low frequencies of occurrence, in the light of recent data. Machine methods of tabulation are well adapted to frequency determinations, and would remove much of the drudgery involved in such reanalysis. This would entail transferring rainfall data onto punch cards, including hourly and daily published amounts, as well as values for shorter periods taken from original recorder charts. The Weather Bureau is now card-punching all current hourly meteorological observations at 500 field stations, and this includes rainfall measurements, but has been unable to undertake processing of the huge backlog of data, except on a very limited scale.

The author has emphasized the distinction between point rainfall as measured by the individual gage, and the volumetric amounts over drainage areas usually required in practice. Figure 41 of the author's paper is a graphic illustration of the errors that might be involved in volume estimates of rainfall made from the ordinary network of gages. The present spacing of rain gages is satisfactory for certain purposes such as determination of long-term precipitation averages in fairly level areas, but has obvious weaknesses in connection with storm studies. It is probably impracticable to provide the entire country with gages as closely spaced as the experimental Muskingum network. However, it is generally recognized that a considerable expansion of the present rainfall network is well justified economically. The Subcommittee on Hydrology of the Federal Interagency River Basin Committee has reviewed this problem, and has recently advocated establishment of approximately 4500 additional rainfall stations.

A report has recently been issued on a joint Thunderstorm Project of four government agencies: Air Force, Navy, National Advisory Committee for Aeronautics, and Weather Bureau. The investigation was mainly directed towards obtaining information relative to flying hazards, but some of the conclusions and facts have important hydrologic implications. It was found that a single, isolated thunderstorm is a rare phenomenon, and that generally there are three or more thunderstorm cells adjacent to each other. The maximum rate of rainfall occurs near the cell core within two to three minutes after the onset of rain. The rain intensity is heavy for five to fifteen minutes, and then slackens off slowly. The total storm duration will vary from a few minutes for a weak cell to almost an hour for large active ones. Small scale land features such as lakes, swamps, hills, and woods influence the areal distribution and frequency of occurrence of thunderstorms. The last conclusion indicates that numerous anomalies may exist in the geographic pattern of rainfall, but remain undetected because of the open network of gages.

The method of maximum possible storm determination outlined by the author conforms to a great extent to practices followed by the Hydrometeorological Section of the Weather Bureau in their studies for the Corps of Engineers. The method is described in detail in the author's references, but it may be of interest to point out the immense amount of work required in the processing and analysis of data. Meteorological data for all the historical storms of record within a large region surrounding the basin of interest are examined to determine whether they can be transposed, and if adjustments are required in the process. The observed rainfall within these storms is extrapolated to an upper limit on the basis of a possible increase in the controlling meteorological factors. The need for such a specialized study depends on the scale of a water-control project, and the question of safety to life in case of failure resulting from a critical storm during and after construction. However, in most cases the cost of such a study is insignificant compared to the total cost of a project and is well justified.

Current proposals for the conservation and control of water within large river basins will make considerable demands on technical knowledge and data in evaluation and planning of projects. The need for an expanded basic data collection and research program to meet those demands is aptly stated in the following excerpt from House Document No. 706, 81st Congress:

"As a nation we cannot afford to run the risk of dissipating our resources by building projects that are not adequately founded on good engineering and scientific facts. It is, therefore, of utmost importance to provide the means for correcting the deficiencies in hydrologic basic data by collecting adequate records of precipitation, stream flow, ground water, quality of water, sediment loads, evapo-transpiration, and all other essential data. Provision should also be made for comprehensive research into the relationships among all these factors, and methods by which the maximum utilization of our water resources may be effected."

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A PANEL DISCUSSION: WATER USE

R. G. SNIDER,* Moderator

I must initially point out that any of the developments which the Conservation Foundation gets into come very largely from the association with experts and it depends essentially upon them. Actually, we are a group who were selected as generalists. Therefore, I can only point out one or two rather obvious truths about the point of view we take.

Today we are to hear from the customers. Up until now, I believe it is fair to say we have been talking about the water problem largely from the supplier's point of view. Now we are to hear from the customers. In business the customer is rather an important individual and all of us are rather eager to hear just how the water customer views the water problem locally and on a national basis. I have been fortunate to read most of the papers and we have a very interesting morning ahead of us. Night before last Dr. Buswell made a rather interesting analogy when he told the story of the hen who was given duck eggs and eagle eggs along with hen's eggs. For those of you who were not there, the analogy was to indicate the breadth of this particular meeting. We have the radar-weather people at a meeting nearby. We have the engineering and the water treatment people, and we have other representatives of other groups. Mr. Veatch yesterday indicated the need, as have a number of other speakers, for greater public understanding. The first step about water problems is to cross a certain number of technical boundaries in an effort to make each specialized group realize how his problems bear on those of his neighbor, but even more important once that step is understood, is the need to make the general public aware that water is of particular significance and cannot be taken entirely for granted. I have noted that, because of an awareness I have toward the lack of misunderstanding, a number of people mentioned their great surprise when they found that water was not to be taken for granted.

In the course of the papers this morning we will see some of the critical points which must be made clear to a very large body of the public before we reach really adequate action on meeting local and regional water problems. For that reason I believe we should bear in mind the rather unusual approach which was taken at the banquet meeting last night of bringing in another in a field entirely separate. Mr. Eichelberger was speaking on the United Nations and deliberately did not talk to himself. So many of the technical readings are directed toward one's technical associates. It is, of course, important if we are to make the progress we need to in water developments, that we go around and talk not to ourselves but to other groups and make them understand what the impact of water is on their daily life.

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INDUSTRY'S WATER PROBLEMS

BY THOMAS J. POWERS*

A glance at the bibliography that Dr. Raymond Hess has prepared for the Water Pollution Abatement Committee of the Manufacturing Chemists' Association reveals at once that industry's water problems are many and varied. A reviewer of all the articles written on this subject during the past five years will readily see that industry's water problems revolve about three general topics:

1. Water quantity
2. Water quality
3. Waste water control

Before we discuss these in general, let's see what an ideal industrial water supply might be.

1. Unlimited volume
2. Constant low temperature, say 60° F.
3. Limited turbidity and suspended solids, say 10 ppm.
4. No dissolved gases O_2, CO_2, H_2S
5. Neither corrosive or scaling at 140° F.
6. Low color, say 10 ppm. Cobalt Standard
7. Devoid of plant and bacterial life
8. Low dissolved solids, say 100 ppm.

If we can assume these standards without argument, we immediately conclude that no surface water is ideal. Subsurface waters can more closely approach the ideal, and are used by industry in immense quantities. But since ground water resources are limited, we naturally arrive at industry's water quantity problem.

It would seem that the problem of water quantity can be attributed to one or more of the following:

1. Improper land use resulting in loss through rapid runoff.
2. Local overexpansion of industry and population in relation to water resources.
3. Improper water use resulting in waste.
4. Cyclic deficiences in rainfall.

Industry, of course, can do little or nothing about rainfall or land use, but I think it can safely be said that industry has overexpanded with respect to its water supply in some areas. Those industries which have tied their water supply to municipal supplies are most unfortunate. Where industry expands, there must follow an expansion of population, and the growing population must inevitably have first call on the municipal supply.

The only immediate help for industries now in a serious position for lack of water seems to lie in conservative use of available water and the substitution of sea water or other less ideal water for heat exchange.

This naturally leads us to the topic of water quality. Industrial plants may be designed to use waters which are unfit for public water supply. On the other hand, certain processes and boiler feeds require condensate or completely demineralized water. Where water temperatures are too high for certain product heat exchange, refrigeration must be employed. Generally speaking, however, water quality is not as all-important to industry as it is to municipalities.

Corrosion and scaling of heat exchange equipment is probably industry's largest problem related to water quality. This problem is so complex and specialized for each individual use that we can only recognize it and admit that the cost to industry as a whole is a staggering sum.

Industry can use water of variable quality. For example, a large integrated chemical plant such as Dow at Midland might use the following waters:

1. 200 MGD of river water, chlorinated for slime control, as general service water furnishing heat exchange, fire protection and some processing. The cost of this water is about $20 per million gallons. This water may vary as follows:

   Turbidity - 15 ppm to 40 ppm
   Temperature - 33° F. to 85° F.
   Dissolved oxygen - 6.0 ppm to 12.0 ppm
   Alkalinity - 150 to 220 ppm (bicarbonate)
   Calcium - 30 to 40 ppm
   Magnesium - 15 to 25 ppm
   Color - 10 ppm to 50 ppm

2. 7.0 MGD of Lake Huron water for feed to ion-exchange units and for special heat exchange. Cost of this water is $110 per million gallons and might vary as follows:

   Turbidity - 0-30 ppm
   Temperature - 40° to 70° F.
   Dissolved oxygen - saturated
   Alkalinity - 90-100 ppm
   Calcium - 15-20 ppm
   Magnesium - 6-8 ppm
   Color - 0-5 ppm

3. 2.0 MGD deep well water for special summer heat exchange. These wells deliver 54° F. water, high in iron, low in oxygen, high in total dissolved solids (1000-7000 ppm). Cost of well water is $20 per million gallons.

4. Double ion-exchange water used for special processing and for boiler make up. 2.0 MGD at a cost of $400 per million gallons.

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5. 1.5 MGD of potable water, softened, filtered, and chlorinated at a cost of $150 per million gallons for sanitary purposes and some processing.

The quality of water available for industry depends on the following:

1. Location geology
2. Land use
3. Upstream municipal pollution
4. Upstream industrial pollution

Of these, the only item within industry's control is the last.

We have briefly set forth the water quantity and quality problems related to industrial use. We firmly believe that industry's engineers can solve most of these problems to maintain and expand our industrial might. It appears, however, that industry's water supplies will, in general, become increasingly costly.

The last item, Waste Water Control, is the most serious problem related to industry's water use today. Despite federal, state and local pressure to minimize pollution of public waters, the fact remains that industrial establishments have not been able to tailor waste waters to available dilution. Surprisingly, this is true even on our largest river systems, and is due in large part to the constant expansion being demanded by our present all-out production program.

One of our larger eastern chemical companies is faced with a water problem, partially with respect to supply but mainly with respect to waste water dilution. With the small dilution available, its waste water quality needs to be superlative, but as yet no means have been found to tailor the waste water to meet available dilution. The answer here may be to pipe area waste waters overland to the sea and perhaps a return pipe line to bring sea water inland would permit this company and other industries to expand in that locality.

A midwestern chemical concern has now expanded beyond its waste dilution limits and must spend upwards of two million dollars to further treat some of its wastes and to develop subsurface disposal for other wastes created by an unprecedented demand for certain products.

The future development of the nation's natural resources will depend, in part, on the ability of industry to meet the present, but sometimes hastily adopted, standards for waste waters.

Industry is the first to admit perplexity concerning the evaluation of stream pollution based on chemical standards. Industry has also been the leader in proposing bio-assays as an adjunct to chemical tests.

Industry, as a whole, has attacked the problem of waste water control stepwise by process change and recovery, by skimming, neutralization or precipitation and settling, followed by oxidation or reduction where proved necessary.

Most significant and encouraging, I think, is the growing recognition by industry that while it must fight to protect its water supply and defend its rights, it must by the same token live up to its responsibilities as a corporate citizen in matters of waste water control.

THE NATIONAL PICTURE

BY WALTER PICTON*

WITH DISCUSSIONS BY A. M. BUSWELL, R. G. SNIDER, C. O. WISLER, H. E. HUDSON, JR., HORACE GRAY, AND J. H. MORGAN

THE GENERAL WATER PICTURE

During this conference and during our panel of today, specialists will present details of interest in their specific fields. As a generalist I will attempt to present the national picture as a whole. Probably more appropriately it could be called a jigsaw puzzle picture since many of the pieces are not yet in their appropriate places.

In this presentation I am not speaking as an official government representative who would base all statements on published statistics. Such data are not yet available with which to complete the present picture and project it into the future. Using, therefore, other related information I have assembled an approximate picture from analyses and estimates of my own.

In years to come I will be able to correct and perfect these estimates as results are available from present endeavors to supply the data needed in this field. I hope that the questions raised with regard to my estimates will aid in stimulating additional research.

ECONOMIC IMPORTANCE

The foundation of the pattern of development of our American industrial and metropolitan growth has been based upon the abundant and economical availability of water. Starting by settlements along fresh water, we expanded along the streams for the benefit from navigation and power. We now depend upon water for agriculture, community life, health and industrial operation.

The development, treatment and delivery of water has grown to rank high in importance in our national economic structure. The present value of our investment in water supply facilities for only the municipal and industrial supplies is in the neighborhood of 20 billion dollars and includes nearly 40 million tons of metals in the form of materials and equipment. These facilities require for their maintenance and operation an expenditure of over 600 million dollars annually and a slightly greater expenditure for annual expansion to keep pace with our growing population and industrial production.

The future development to satisfy our increasing water requirements, in order to be successful, must be based on judicious planning. Our problems are principally those of natural water availability, economical development and balanced use.

BASIC USES

For present consideration it is convenient to consider water use in five major categories. From these categories I have eliminated navigation, flood control, hydroelectric generation of power and recreation as nonconsuming uses. I have also eliminated water use for nuclear fission products and military operations since such information is classified. The uses to be considered are:

(a) Irrigation (federal and nonfederal).
(b) Rural (farm and rural nonfarm).
(c) Municipal (or public utility).
(d) Industrial (private self-operated).
(e) Steam power (electric and railroad).

In these categories I have estimated the uses of water for the period from 1900-1950 and made a projection for 1960, Figure 54. The important points are:

(a) Irrigation. This includes primary and supplementary supplies from both surface and ground sources. The acreage irrigated has increased from about 7.6 million acres in 1900 to around 25 million acres in 1950. Likewise, the daily average water use has increased from about 25 billion gallons daily to 100 billion gallons daily in 1950.

(b) Rural. Individual water supplies for rural farms and homes are obtained largely from ground water sources. This use has increased from about 2 billion gallons daily in 1900 to nearly 5 billion gallons daily in 1950.

(c) Municipal. Municipal water supply systems which in 1900 were about 3,000 in number and served less than 30 million people have grown to nearly 16,000 systems serving nearly 94 million people in 1950.

(d) Industrial. The privately owned self-operated water supplies of the larger basic industries, such as steel, oil refining, aluminum, paper and pulp products, textiles, plastics and synthetics have grown with our increase in industrial production. Their use was about 10 billion gallons daily in 1900 and has now increased to about 46 billion gallons daily in 1950.

(e) Steam power. Production of steam-generated electric power and the steam operation of railroads have increased their use of water from about 5 billion gallons daily in 1900 to about 35 billion gallons daily in 1950.

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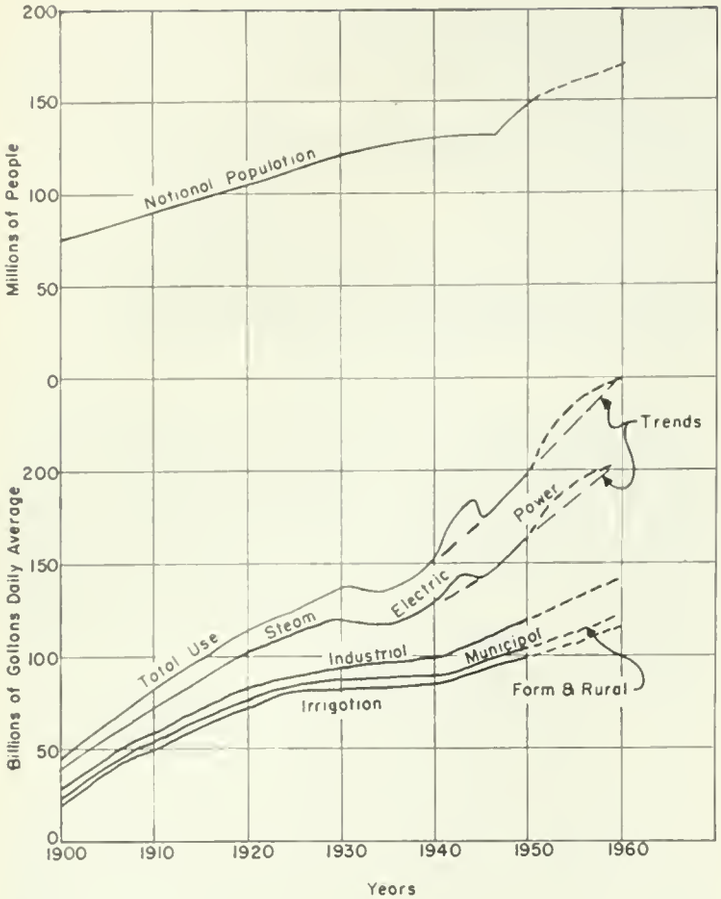

FIG. 54. —GROWTH OF WATER USE IN UNITED STATES, 1900-1950.

Trends of these uses indicate an increase of use much more rapid than our increase in population. The average daily per capita use of municipal water supplies has increased from less than 100 gallons per capita daily in 1900 to 150 gallons per capita daily in 1950. The total of all of these categories has increased from 45 billion gallons daily in 1900 to about 200 billion gallons daily in 1950. In this half century, water use has risen to 450% while the national population has only reached 200%. On a per capita basis the increase is from 600 to over 1,300 gallons per capita per day with indications of continuing increase in the future.

OUR NATIONAL WATER RESOURCES

Let us review our natural water resources that must supply all of these uses. The source of all water resources is precipitation, which varies geographically from a very low in parts of the West to very high in the Pacific Northwest and south Appalachian areas. The estimated annual average for our nation is about 30 inches. After depletions by direct evaporation, evapo-transpiration by plants, and our civilization's consumptive uses, about 27% of this precipitation is returned to the oceans by surface and underground drainage. This residual is the volume of water upon which we are dependent for our future growth.

There are, however, many limitations on the consumptive use of this residual. A large portion of this passes into the sea as flood flow in a very short period of time. The development of availability through stream regulation will have economic limitations which may preclude the development beyond a constant availability of probably half of the present outflow. From this firm flow reserves must be deducted for navigation and for use in transporting our liquid waste. This would indicate as available for our consumption for future growth about 310 billion gallons daily as compared with our present consumption of approximately 80 billion gallons daily average.

POTENTIAL DEVELOPMENT

We must recognize, however, that the over-all viewpoint does not present the picture in proper perspective. We cannot add together all of our water resources and assume that this quantity is available for our entire national growth as we might do with our capacity for steel production. Water cannot be transported long distances economically. Therefore, our national picture must be composed of separate studies of each local area and its potential development.

There are local variations in climate and precipitation. These and other factors cause a wide variation in the natural ground and surface water availability. Some localities are abundantly endowed and others are seriously deficient. Added to this are many water critical areas due to overdevelopment, such as overdraft of ground water and growth of water-using activities beyond the dependable yield of existing surface water developments.

As an aid to the solutions of these problems, research is now underway which may in time be of great value to us. Samples of these are progress in artificial inducement of precipitation and demineralization of saline waters. Advances have been made in water development by impoundment and in watershed development to increase the dependable yield. The artificial inducement of ground water storage may be found feasible for large scale operations which could store flood water underground with minimum loss by evaporation.

RECENT PROGRESS IN ASSEMBLY OF BASIC DATA

Our progress in assembly of basic data is well known in the areas of standard programs such as the collection of data on precipitation, records of stream flow by gaging, records of ground water

levels in wells and the evaporation from water surfaces in selected areas.

In addition to these standard programs, I have had the opportunity to participate in the encouragement and sponsorship of other recent programs and assemblies of data which are to be of high value to us in the future.

The Division of Stream Pollution Control of the Public Health Service is now publishing, as rapidly as their budget will permit a series of river basin reports which give a summary picture of the quality of our streams with respect to pollution. They are following these with area reports known as sub-basin reports, which will contain information which they have been able to gather up to this time.

The Water Resources Division of the Geological Survey has been engaged upon several projects that promise to provide data of very high value for water resources planning. One of these is the assembly of data to present a summary of our national water resources. Recent pamphlets have presented summaries of our water resources such as the stream flow of our large rivers, a list of our major reservoirs and their capacities, and summaries of our ground water problems.

Another publication, of especial interest to me, presented the estimated water use in the United States (Geological Survey Circular No. 115). This tabulates by states the average water use in 1950 by categories similar to those I have previously defined. This is a valuable addition to our information on water utilization.

For practical application in water planning, the Geological Survey is now engaged in making over-all studies of water resources and water utilization in selected local areas. These areas may include a metropolitan district within a radius of 20 or 30 miles or a complete river sub-basin, as may be determined appropriate. These studies will summarize the natural water availability, both surface and underground, the present municipal, industrial, irrigation and other water uses, and give appropriate analyses and indications of the potential water supply development for the area. In parallel with the Public Health Service sub-basin reports, these will provide a fairly complete picture of each of these local areas.

Another important program is being carried on by the Geological Survey's Water Utilization Section by making surveys and assembling data on water use by selected industries. This will provide details not previously available on the water requirements, for such purposes as cooling, washing, processing and boiler feed, for the production of many products. In making this survey by personal visit, it is sometimes possible to obtain special information on the water requirements for various stages of the process and different methods of production, the ability to re-use water, and the net consumption. By the selection of plants in different localities, the adaptability to use of ground or surface water and the ability to use saline water for cooling can be studied. At some later time this information will be properly consolidated and summarized without revealing individual plant data and can then be released for public use.

Other federal publications of data with which you may be familiar are the following:

(a) The Public Health Service Census of

Table I

Estimate of Water Use in the United States, 1900-1950-1960
Average Daily Use in Billion Gallons per Day

Categories of Use	1900	1920	1940 Pre-War	1943-44 War Peak	1945 War End	1950 Post-War	1960 Future
Irrigation	25	75	85	90	94	100	115
Rural (and farm)	2	2	3	3	3	5	6
Municipal (public)	3	6	10	12	12	14	20
Industrial (private)	10	20	30	40	35	46	65
Steam power (electric and railroad)	5	10	25	40	30	35	45
Total	45	113	153	185	174	200	251

Public Water and Sewage Works Facilities. This information is assembled annually although no publication has been made since 1945. It is hoped that another publication will be made in 1952.

(b) Recently, cooperative analyses have been made by the Public Health Service and the Bureau of Labor Statistics of the quantities of material and equipment required for the construction of water and sewage works.

(c) The report of the President's Water Resources Policy Commission indicates the administration's interest in the development of our water resources. However, in covering so broad a subject, very little space was given to specific problems in the municipal and industrial water supply fields.

(d) The President's Material Policy Commission is now completing its report. I am hopeful that their report will give more attention to the problems in future development of water supply for industrial expansion.

Certain nonfederal surveys and reports have added materially to our fund of water resources knowledge. Among these are a study of the Conservation Foundation of groundwater, the cooperative report of the National Association of Manufacturers and the Conservation Foundation on the use of water in industry, and the publications by several of our states of assembled valuable information on the municipal and industrial use of ground and surface water within their states.

OUTLOOK

It is the problem of this and other Water Conferences to see to it that this interest in water resources, water supply development, and water utilization be maintained. The research now underway must be continued and research initiated in other phases in order to provide needed information which at present is insufficient in many areas.

This basic data can be used for intelligent balanced planning of development and use of our water resources and for projections of our requirements for various categories of use. Such planning must of necessity be made by natural areas with collaboration in such planning with adjacent areas and larger regions. The responsibility should be local but with the full benefit of the advice that can be obtained from federal research and coordination in resolving any special problems.

Our future development for growth may depend upon the wisdom and foresight of these many local areas in the development and use of our water resources. Properly guided by adequate basic information, we can hope to attain a well balanced economy locally, regionally and nationally by the optimum utilization of our water resources.

DISCUSSION

A. M. BUSWELL. —One or two points come to mind in connection with this discussion because, as you know, the problem of water resources is the problem of the Illinois State Water Survey at the present time. It is somewhat later than we think with respect to our long-time planning in Illinois and presumably in the rest of the country. An estimate made by Farrington Daniels[1] is that there is "power available in the world to support two thousand times our present population." Our estimates are that in Illinois the present total water available could accommodate, with our present methods, ten times our present population. The first shortage that will be encountered will be that of water supply.

What is the long-term big source of water? That, I think, was touched upon by Dr. Conant[2] in addressing the recent meetings of chemists in New York. Dr. Conant predicted that we would be obtaining water from the ocean, not by chemical treatment but by evaporation. It happens that in war research work the State Water Survey's laboratory did considerable work along that line. The best that could be obtained was a ratio of about three and a half pounds of water to one pound of chemical.

The big source of water is atmospheric moisture. There are comparatively few who realize the relation between the amount of rain which falls and the amount of moisture in the atmosphere. We have had some calculations made which show that, when it rains, a relatively minor fraction of all the moisture in the atmosphere falls to the earth, and a considerable fraction is immediately evaporated back into the atmosphere. During the drouth of 1947 there were 20 days when, as far as the moisture and cloud formation were concerned, there were favorable conditions for heavy rains. While I am not a meteorologist, I feel that atmospheric moisture can be a big source of water supply. From a long-time standpoint, it is our large unexplored source of water supply. It, of course, results from evaporation from the ocean by solar means and is brought inland by means of winds. There is some reason to hope that if we should decide to take the moisture out more rapidly, the process of replenishment might be accelerated.

Our modern developments in industry require tremendous amounts of water at very low cost. Many industries must have water at a cost of less than two cents or down to a fraction of a cent. To get costs down, we need to put into effect some of the things we already know.

In Illinois we have a large amount of undeveloped surface water. We have hundreds of good reservoir sites which could be developed, and we have a good many places where recharge of ground water is feasible.

R. G. SNIDER. —I am glad you brought up that point. The Conservation Foundation is considering undertaking the sponsorship of a preliminary engineering survey dealing with the problem of converting salt water to fresh.

C. O. WISLER.[3]—Several of our speakers this morning have referred to the possibility of augmenting our water supplies through the practice of rainmaking. It is a well established fact that something like approximately 80 per cent of the water that falls as rain had its origin in oceanic evaporation. The exact per cent varies. The principal source of our water vapor is from large air masses and in this area it is from the Pacific air masses. Suppose rainmaking is developed in the arid west to the extent of its possibilities. What would happen here in the central United States? I am thinking of the legal aspect and the possible involvements that might follow. I believe that it has been determined that, although 80 per cent of the entire rain that falls comes from oceanic evaporation, yet in no part is that per cent less than 50. Here we have in Illinois about 35 inches of rainfall. Suppose that 50 per cent that comes from oceanic evaporation has been removed from the air before it gets to Illinois?

H. E. HUDSON, JR.[4]—The studies to which Dr. Buswell referred in his comments during the panel discussion on water use, are parallel to those published by Dr. Benton and indicate we are receiving about five per cent of the moisture that passes over the state. The studies we made for the years 1947 to 1949 indicate we were receiving precipitable water about half of the time it was available.[5] We may consider precipitable water as that which might be converted into rainfall by artificial means. It is possible that, had others taken some of that water, our take would have been reduced. After all, cloud seeding will not be done over a state as a whole, but in particular areas. Illinois is not worried. It appears that we can get water at certain points; meanwhile, we have much to learn about the process.

HORACE GRAY. —If things of this sort come to pass, I think the New River doctrine would solve the problem. It would extend the public control from surface and ground to water in the clouds.

J. H. MORGAN.[6]—I am under the impression that our present atmospheric moisture supply comes from the Gulf. There is plenty of water south of here.

[1]Science, Jan. 25, 1949.

[2]President, Harvard University, Cambridge, Mass.

[3]Professor, University of Michigan, Ann Arbor, Michigan.

[4]Engineer, Illinois State Water Survey, Urbana, Illinois.

[5]"Radar and Rainfall," Report of Investigation No. 3, Illinois State Water Survey, 1949.

[6]District Engineer, United States Geological Survey, Champaign, Illinois.

WATER USE CONSIDERATIONS IN THE FEDERAL WATER POLLUTION CONTROL PROGRAM

BY PAUL W. REED*

The use to which the waters of the nation were to be put have always been a primary consideration in the administration of the Federal water pollution control program as well as in the water pollution control programs of most of the states. In fact the basic law, Public Law 845, states in Section 2:

"The Surgeon General shall, after careful investigation, and in cooperation with other Federal agencies, with State water pollution agencies and interstate agencies, and with municipalities and industries involved, prepare or adopt comprehensive programs for eliminating or reducing the pollution of interstate waters and tributaries thereof and improving the sanitary condition of surface and underground waters. In the development of such comprehensive programs due regard shall be given to the improvements which are necessary to conserve such waters for public water supplies, propagation of fish and aquatic life, recreational purposes and agriculture, industrial, and other legitimate uses."

Public Law 845 was approved June 30, 1948, but appropriations were not made available by Congress for its administration until July 1949. Following the passage of the law, the U. S. Public Health Service established 10 offices in the major drainage basins of the country to carry out its functions. The Upper Mississippi and Great Lakes Drainage Basins Office in Chicago was established on July 1, 1949. These offices were staffed, as rapidly as personnel became available, with two or three sanitary engineers, an aquatic biologist, a draftsman, and the necessary clerical assistance.

One of the first activities of the basin office was to obtain information upon which to base the comprehensive reports called for in the law. The states cooperated in this endeavor by supplying basic data on water use, sources of pollution, existing treatment facilities, needs, construction progress, etc., and the basin office compiled the statistical data from these basic data. Fifteen major drainage basin reports based on these data are in various stages of development at the present time. The reports on the Tennessee and Missouri are already out and others should be released soon.

The Federal law also directed the Surgeon General to "collect and disseminate information relating to water pollution and the prevention and abatement thereof" and "support and aid technical research to devise and perfect methods of treatment of industrial wastes ...."

In keeping with the provisions of these sections, part of the grant funds is made available to states for research or special studies. Typical of these projects is the study of the lagoon method of the disposal of sugar beet wastes carried on at Moorhead, Minnesota, under the sponsorship of the Minnesota Water Pollution Control Commission.

Wisconsin is sponsoring a study of industrial waste treatment plant design standards in behalf of the Upper Mississippi and Great Lakes Boards of Public Health Engineers. While it is realized that industrial wastes do not conform to any standard and, therefore, are not susceptible to the application of standard design factors, it is hoped to develop some guide which can be of assistance to those designing treatment facilities.

Upon the formation of the Illinois-Missouri Bi-State Development Agencies, the States of Illinois and Missouri suggested that this agency be utilized to sponsor a pollution study in the St. Louis - East St. Louis metropolitan area. This study, which was needed for many reasons, was specifically recommended because the commercial fishermen below the metropolitan area complained that pollution was rendering the use of the stream unfit for this purpose. At the present time a joint survey is being made by the Bi-State Agency, Illinois, Missouri, and the Public Health Service.

The Environmental Health Center at Cincinnati has a number of research projects in progress on matters related to water pollution control. Some of these are: biological indicators of pollution; concentrations and analysis of minute quantities of organic chemicals causing taste and odor in public water supplies; basic data on nature and effects of various industrial wastes. And there are many others.

Unfortunately, the average individual looks upon stream pollution abatement programs as "save the fish" programs without realizing that this is but one use of water. They fail to appreciate the very vital part that clean water plays in the industrial productivity in which we as a nation have so much pride. Few people realize, for instance, that steam generation of electric power requires from 52 to 170 thousand gallons of water per 1,000 kilowatt hours; rolled steel, 6 to 110 thousand gallons per

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ton; or aviation gasoline, one million gallons of water per 1,000 barrels, to mention a few.

Water is vital to man in many ways. Even the food he uses requires water to grow, and when grown requires more water for processing. The canning of vegetables, for instance, requires from 60 to 16,000 gallons per 100 cases (24 No. 2 cans). Five gallons of water are required to process each gallon of milk, and 6,000 gallons are required to process one ton of meat.

Little do we realize on a cold winter night as we fire up our furnaces that 200 gallons of water are needed to wash each ton of coal, or if we want a cocktail that 80,000 gallons of water are needed for the manufacture of each 1,000 gallons of whiskey.

Nothing has been said so far about the use of water for public water supply because it should be apparent that an adequate supply of clean water for this use is vital. In the past in some regions we have relied heavily on our ground waters as a source of public and industrial water supplies, and as a result many persons could not become seriously concerned about the pollution of surface waters. In recent years we have heard much of the shortage of ground water in critical areas. While we do not have the time to consider this problem in detail, it is evident that if we continue as a nation to need more and more water and if available ground waters decrease, conservation of our surface waters becomes increasingly important.

Our water resources are the very backbone of our entire existence so we should not waste those resources. We should also remember that the destruction of water by pollution denies our use of them just as surely as though they were physically removed from us.

INDUSTRIAL CONSERVATION METHODS FOR RE-USE OF COOLING WATER

BY HOWARD E. DEGLER*

Water is America's No. 1 natural resource; it is as essential to industrial life as to animal and plant life. It is a universal solvent, catalyst, tireless medium for transport of materials, disposal of wastes, cooling agent, dispersive medium, cleansing vehicle, and vitally essential for production and distribution of process heat and power.

During the past decade the average peak summer demand for water in U. S. A. cities has increased from 100 to 200 gallons per day per person. This has resulted in present-day problems of (1) inadequate sources of water, (2) inadequate purification and distribution systems, or (3) inadequate sewer disposal and waste systems.

Water problems may be divided into five groups, ground water, surface water, pollution, recharge and re-use. In general, the quantity and temperature of water available are of the utmost importance, far outweighing considerations of quality. But the quality of the water supply cannot be entirely ignored because dissolved salts, gases or other impurities can corrode or foul heat exchangers. Depletion of ground water has made necessary the use of more and more surface water, which in turn may be polluted and requires treatment before distribution.

Selection of the most feasible water-cooling method becomes more important as power demands increase and industries expand their capacity. There is plenty of water, at a price. Since the doubled demand for water during the past decade, underground sources in many areas have become dangerously depleted by overuse. Runoff water is that which remains on the earth's surface after the prior claims of evaporation, transpiration of plants and absorption by the ground have been satisfied.

COOLING WATER REQUIREMENTS

Irrigating an acre of oranges requires 800,000 gallons of water per wetting. Producing one kilowatt of electric power takes 3000 gallons of cooling water per day, making one ton of ice requires over 6000 gallons of cooling water per day, and the midsummer comfort cooling of a large theater requires 75,000 gallons per day. One large paper mill uses 25 million gallons per day, more than the total water pumped for an average American city of 200,000 persons.

Many processes require huge quantities of cooling water; Table I lists 15 examples. Great as the total water requirements are, they are much less than the total amount of rainfall and less than the smaller amount available for use from streams and underground sources. However, the availability is continuously decreasing and the need for water conservation and regulation becomes more imminent.

WATER SYSTEM EXPANSION COSTS VS. COOLING EQUIPMENT COSTS

Kansas City uses 5% of its annual water output for air conditioning, but the latter use accounts for 15% of the maximum daily demand. Accordingly, 15% of the water supply capacity is earmarked for air conditioning use but less than 5% of the income is received from sales of air conditioning water since large quantity use results in reduced rates. Kansas City has about 75 tons of air conditioning per 1000 population. Very few water rate schedules were planned to cover a class of customers

Table I

Water Use in Typical Industries

Product	Lb. of Water per Lb. of Product
Aluminum	500
Ammonium sulphate	700
Butadiene	1200
Gun powder	400
Hydrogen	2500
Lactose	800
Linen	700
Oil refining	500
Paper	200
Rayon	800
Rubber (synthetic GR-S)	2500
Soda pulp	300
Steel	250
Textiles	350
Wool	500

*Technical Director, The Marley Company, Inc., Kansas City, Kansas.

who require little or no water for 8 months per year, but wanted 10 to 15% of the total output of the annual peak day.

The water requirements for air conditioning may increase so greatly in some cities as to require an enlargement of plant and a water main expansion program. Revenue bonds would be voted to obtain funds for construction, but servicing the bonds would probably require a considerable increase in water rates. These increased rates might cause many of the air conditioning customers to install water-cooling equipment, thus reducing water demands so much that the expanded water facilities would not be needed after they were built.

Water system extensions, plant, mains, etc., cost approximately $700,000 per million gallons of peak day capacity, or about $3500 for 5000 gallons supplied on a hot summer day. A 5-ton refrigeration unit using 2 gpm. per ton and operating 8 1/3 hours per day would require 5000 gallons; hence, the water utility investment will approximate $3500 for the cooling water required by 5 tons of refrigeration. Water cooling equipment, to adequately cool a 5-ton refrigeration, could be obtained at considerably lower cost, hence there is definite basis for city ordinances requiring the use of water conservation units instead of overtaxing the water system and the sewer system.

Some U. S. A. cities have adopted ordinances to control the maximum rate at which water can be taken from city water mains for use in "once-through and to the sewer" for cooling air conditioning systems. Generally, these regulations prohibit the use of city water in systems having a capacity exceeding 5 tons of refrigeration, unless water conservation equipment is installed. However, such regulations should also be applied to other large peak load uses of water for swimming pools, golf courses, irrigation, truck gardens, greenhouses, laundries, etc.

COOLING METHODS FOR WATER RE-USE

"Once-through and to the sewer" use of cooling water is wasteful. In many applications, the same water can be used for additional service and/or continuous re-use. The application and selection of the proper type of cooling equipment becomes more important as homes, hotels, stores, theaters, restaurants, etc., increase their demands for water and as power demands increase, chemical processes are developed, and industries expand their water usage. It has been predicted that in another 25 years, sea water will be processed to meet fresh water requirements in many coastal cities.

During the past 40 years, the progressive improvement of methods for cooling and subsequent re-use of water have been utilized in the following sequence: (1) rivers, lakes, wells, and cooling ponds, (2) spray ponds, (3) evaporative condensers, (4) natural-draft water-cooling towers, (5) forced-draft cooling towers, (6) induced-draft cooling towers, and (7) dry-surface coolers (air-cooled exchangers).

Previous to 1920 large water-cooling systems were used only with steam-electric generating stations, usually located on the banks of rivers, lakes, or artificial cooling ponds. In some installations it was expedient to use spray ponds. Refrigeration and ice-making constituted the second largest usage, with roof-located natural-draft towers and spray ponds.

Since the year 1920, numerous new applications have arisen, others grown; and with this shift, requirements have become increasingly varied and exacting, necessitating refinements and specialized adaptations of water-cooling equipment. The principal demand for large water-cooling systems in recent years has been from the petroleum industry, steam power plants, process industries, and chemical plants. Refrigeration, air conditioning, and engine-jacket cooling, each employ a large percentage of the medium-size and small cooling units installed today.

Progress of the cooling equipment industry is apparent when it is realized that for a definite need, a modern cooling unit requires only one square foot of ground area as compared to 50 square feet for the spray pond and about 1000 square feet for a natural cooling lake or pond. The continuous demand for a more compact design, better construction, lower cost, larger capacity, greater flexibility of operation, independence of atmospheric uncertainties, and improved all-around performance has resulted in the modern induced-draft water-cooling tower.

PRINCIPLES OF EVAPORATIVE COOLING

The cooling of water in spray ponds, cooling towers, evaporative condensers, etc., is accomplished by moving air across exposed water surfaces; the evaporation of about 1% of the water will cool the remaining 99% of the water 10 to 12 degrees F.

The main difference between various designs of cooling equipment is the method of air circulation, whether it be natural circulation by wind or mechanical circulation with fans. Other differences would be the manner in which the water surfaces are exposed to the air.

Spray Ponds. A spray pond consists of a number of spray nozzles over a water-collecting basin. Nozzles spray droplets—not mist, because cooled water must drop into the pond and not be blown away. Louver fences on the pond's leeward side keep water from being carried away. These enclosures are a

"must" in restricted areas and on roofs.

Spray ponds are best for large capacity service where efficiency is not too important and moisture drift is not objectionable. These ponds are cheaper although basin costs and pumping requirements are high.

Atmospheric Towers. These spray-filled water-cooling towers, Fig. 55-1, are used (1) when equipment served can stand a few degrees rise in the cold-water temperature at low- or zero-wind velocities, (2) where drift from the tower is not objectionable, and (3) when the tower can be placed so the wind is not cut off by buildings, trees, etc.

Actually, these towers simulate the spray-pond design (one-fourth that of an equivalent spray-pond area) but with nozzles at top and a high louver fence. The nozzles spray downward and the louvers are always wet, hence adding to the water surface exposed to the cooling air.

Mechanical-Draft Towers. These towers, Fig. 55-2 and 55-3, employ a vertical casing made of redwood, metal, asbestos-cement, or masonry. Hot water is distributed near the top in a variety of ways and falls to the cold-water collecting basin. This water passes through the air that is circulated from bottom to top by forced- or induced-draft fans.

The inside of a mechanical-draft tower may be filled with a spray of water droplets from nozzles or packed with wood filling on which water cascades from top to bottom. In many cases, a combination spray-filled and wood-filled design is used, Fig. 55-3.

The air contacts the hottest water just before leaving the tower. Because it passes against the flow of water, a given quantity of air thus picks up more heat than the average equal quantity of air on natural-draft equipment. So less air is needed to cool the same amount of water. As air is supplied by fans, air quantity must be held to a minimum for low operating cost.

Redwood slats are laid through the tower both horizontally and vertically. Water drops from piece to piece of wood filling. As air moves upward or across, it strikes a large wetted surface, repeatedly breaking up the falling drops and providing new drop surfaces whose combined areas are several times the wood-fill area.

The efficiency of this type tower is improved by increasing the filling, height, area, or air quantity. Increasing height increases the time air is in contact with water, without any more fan power.

Induced-draft towers do not depend upon wind velocity, hence it is possible to design them for more exacting performance. They require less space and less piping than atmospheric towers. Pumping head varies from 11 to 26 feet. Improved over-all plant economy from lower water temperature usually offsets the added operating expense and initial cost compared to atmospheric towers.

Double-Flow Towers. The demand for compact design, better construction, lower cost, larger capacity, more flexible operation, and improved all-around performance has produced the double-flow induced-draft tower (Fig. 55-4), sometimes called a cross-flow tower.

Air flow is horizontal with fans centered along the top. Each fan draws air through two cells paired to a suction chamber that is partitioned midway beneath the fan and fitted with drift eliminators which turn the air upward toward the fan outlet.

Double-flow towers use a low pumping head, varying from 11 to 26 feet; this type unit occupies less than one-twentieth the area of a spray pond for equivalent service. Operating advantages are: (1) horizontal (cross-flow) air movement as water falls in a cascade of small drops over the filling and across the air stream. This offers less resistance to air flow, therefore a lower draft loss. (2) It has longer air travel than conventional design. (3) The open water-distribution basin is accessible for cleaning during operation. (4) It has a close-spaced, wood, diffusion deck under the water basin for uniform water distribution to the wood filling. (5) Water loading in most cooling towers has a maximum of 6 gpm. per square foot caused by the blanketing spray effect. Heavier loadings, up to 10 gpm. per square foot, are possible in double-flow towers for large-capacity cooling.

Small Packaged Tower. For cooling load requirements of 3 to 50 tons of refrigeration as required by many air conditioning installations, packaged water-cooling units (Fig. 56-1) are generally used; in this design the fan is at one end to provide horizontal or cross-flow air movement. Frequently used conditions for refrigeration units are 95° F. hot water, 85° F. cold water, and 78° F. wet bulb temperatures, with 3 gpm. of circulating water per ton of refrigeration.

COOLING TOWER COSTS

Small Tower Costs. Table II shows the tower costs for units providing the cooling requirements of 5 to 150 tons of refrigeration. For these sizes the total annual cost of owning a water conservation unit averages half the cost of city water used once for cooling and then wasted (see the 50-ton installation in Table II). In general, amortization of the cooling tower can be effected in 2 to 3 years from the savings in water costs for units of 50 tons or larger capacity. This estimate would be even more justifiable for longer cooling seasons and where rates exceed $1.25 per 1000 cubic feet (17¢ per 1000 gallons), as is the case in many cities.

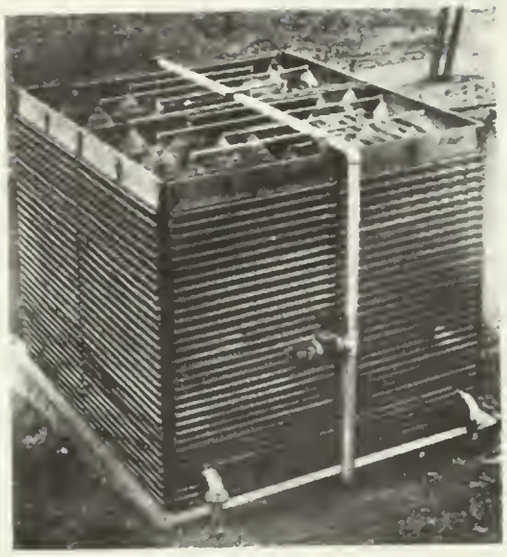

FIG. 55-1. —Spray-filled natural draft water-cooling tower. Note valve on supply pipe for use as winter by-pass.

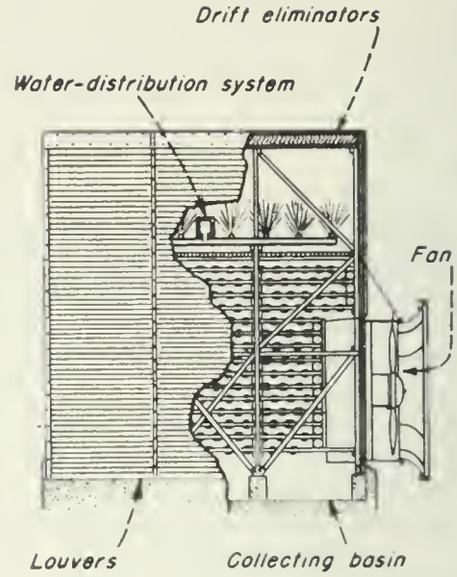

FIG. 55-2. —Forced-draft water-cooling tower. Used very little today.

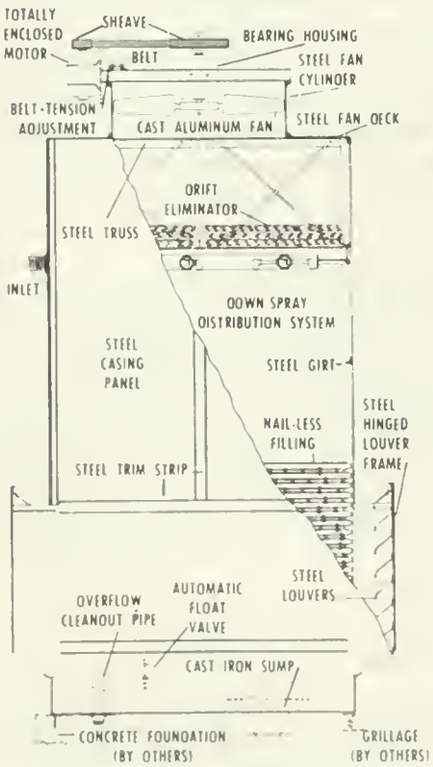

FIG. 55-3. —Induced-draft counterflow water-cooling tower, "conventional" industrial type of medium capacity (800 to 5000 gpm).

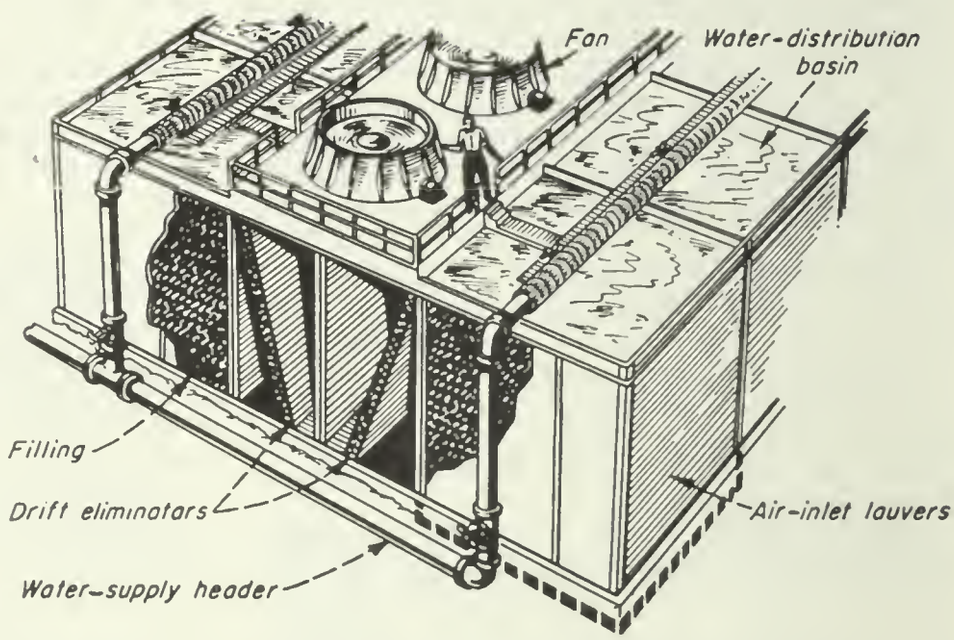

FIG. 55-4. —Induced-draft crossflow (double-flow) water-cooling tower; large capacity (5000 to 100,000 gpm).

FIGURE 55

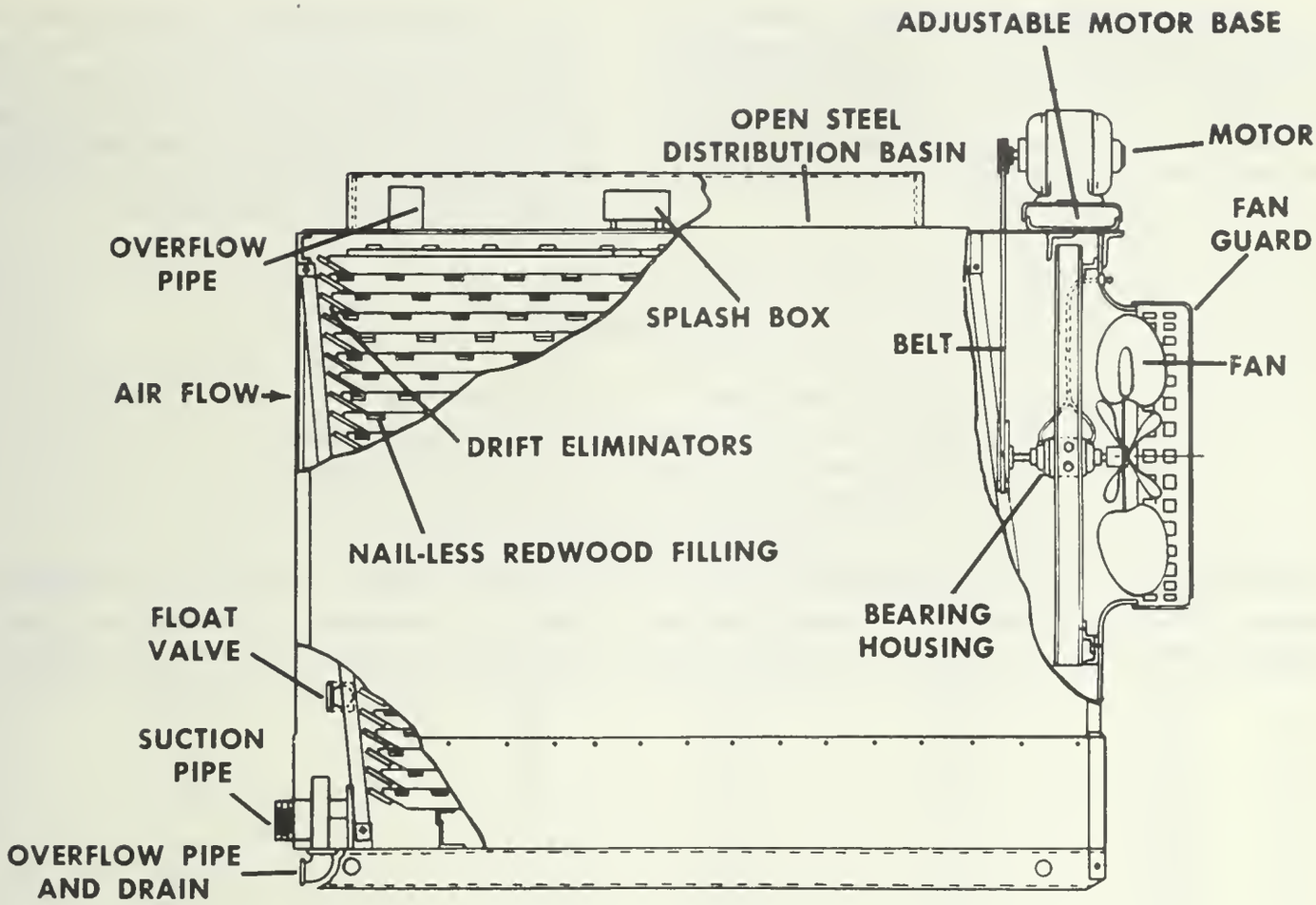

FIG. 56-1. —Small packaged water-cooling tower, induced-draft type; 2 to 60 tons of refrigeration.

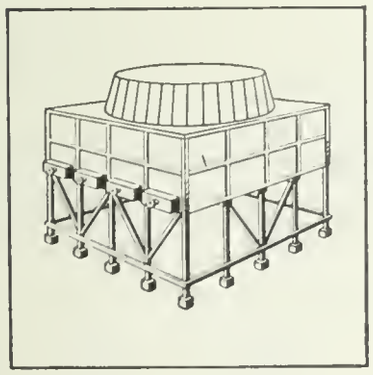

FIG. 56-2. —Induced-draft air-cooled heat exchanger.

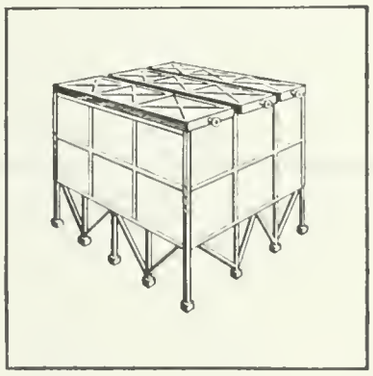

FIG. 56-3. —Forced-draft air-cooled heat exchanger.

FIGURE 56

Large Tower Costs. Table III presents the characteristics and costs of two representative sizes of industrial water-cooling towers for the years 1925, 1932, 1941 and 1950; careful study will indicate that improved designs have continuously reduced costs during this 25-year period, despite the recent inflationary period.

The increased costs of labor and materials for modern water-cooling towers have been offset by better engineering as reflected by continued improvement, simplification, and closer adaptation to the specific requirements of each application, and developing every opportunity for wise reductions in the cost of production and operation.

Table II

Tower Costs vs. City Water Costs

3 gpm. per ton of refrigeration with cooling tower
1 1/2 gpm. per ton with city water, once through
1500-hour per year operation, 95°-85°-78° F.
$1.25 per 1000 cubic feet of water

Size of Tower (tons)	Installed Tower Cost	Interest and Depreciation (15%)	Water treatment, 25¢ per ton per month	Operation of Fan and Pump	Water Make-up at 2%*	Over-all Tower Cost per Year	City Water Cost per Year
5	$ 400	$ 60	$ 5	$ 20	$ 5	$ 90	$ 113
10	600	90	10	28	10	138	225
15	750	113	15	40	14	182	338
25	1150	173	25	66	23	287	562
35	1400	210	35	80	32	357	786
50	1800	270	50	110	45	475	1125
75	2800	420	75	160	68	723	1685
100	4200	630	100	210	96	1036	2250
125	4600	690	125	240	113	1168	2810
150	5200	780	150	260	135	1325	3380

*Includes evaporation, drift, blow-down and leaks.

Table III

Characteristics and Costs of Water-Cooling Towers, 1925 to 1950

95° F. hot water, 85° F. cold water, 75° F. wet bulb

Capacity	5,000 gpm. (3,000 Kw)				20,000 gpm. (12,500 Kw)			
Years	1925	1932	1941	1950	1925	1932	1941	1950
Base, ft.	22 x 95	30 x 72	30 x 80	36 x 36	36 x 220	26 x 240	40 x 240	60 x 60
Height, ft.	40	36	26	20	40	36	26	26
Area, sq. ft.	2,112	2,160	2,400	1,000	8,420	8,640	9,600	3,600
Fans, number	6	6	4	2	22	24	10	2
size, dia., ft.	11	11	13	12	12	11	16	22
bhp, ea.	28.4	19.2	24	25	30	19.2	38.4	70
bhp, total	170.4	114.9	96	50	660	460.8	384.0	140
Weight, dry, lb.	280,000	260,000	240,000	90,000	960,000	940,000	900,000	290,000
Pump head, ft.	27	27	20	14	28	28	20	26
Total cost	$22,000	$20,000	$19,000	$18,500*	$78,000	$71,000	$67,000	$62,000*
Cost per Kw	$7.30	$6.70	$6.30	$6.17	$6.30	$5.70	$5.40	$4.95

*1943-45 costs were 60 percent of these amounts.

AIR-COOLED EXCHANGERS

The air-cooled exchanger is a device for cooling or condensing fluids by circulation of the fluid through finned tubes and forcing of atmospheric air across the coil sections. This dry-surface cooler is basically a noncontact convection-type heat exchanger and requires no water for cooling. One arrangement for a dry-cooler is the induced draft unit (Fig. 56-2) with the fan mounted on top, discharging the air upward and inducing air across a horizontal bank of coils mounted below the fan. Another arrangement is the forced-draft (Fig. 56-3) with the fan discharging air upward across a horizontal bank of coils mounted above the fan.

The air-cooled exchanger may be used in preference to a water-cooling tower for applications of "high-level heat removal" where temperatures of the fluids to be cooled are above 130° F., and where water is scarce, expensive, and/or badly polluted. Normally, a cooling tower is more economical, but the cost of the air-cooled exchanger decreases relative to that of the tower as the temperature of the fluid-to-be-cooled rises. Both units have their own applications and in some instances either type may be used.

CONCLUSION

Water-conserving devices are definitely assisting in the efforts to sustain water supplies in many sections. However, population increases, added irrigation requirements, rapid industrial expansion, and hundreds of other lesser applications, in many sections are increasing the demands for water more rapidly than is reflected by the rate of installed capacity of cooling equipment. In such areas, restrictive legislation on the use of water is preferable to increase in water rates or costly added facilities because of the economy of present-day water-cooling equipment. Also, the re-use of any coolant within a given heat cycle is desirable because of the improved control of the fluid and correlated process.

INSTITUTIONAL CHANGE AND WATER RESOURCE UTILIZATION

BY HORACE M. GRAY*

When the State of Illinois constructed a modern headquarters and laboratory building to house the State Water Survey it thereby gave physical expression to the common concern of its citizens for improved control and utilization of the state's water resources. At the same time, it was an act of faith, based on the deep-seated conviction that free men armed with scientific knowledge can master the natural forces of their environment and use them for the maximum benefit of mankind. This dedicatory Conference, with its array of distinguished scientists, engineers and administrators, likewise symbolizes this faith in the beneficence of scientific method applied to resource utilization. By the pooling of knowledge and the interchange of views, this conference can, and I am sure will, point the way toward making that faith a living reality.

But, if the vast store of knowledge accumulated by the devoted labors of the scientists and engineers is to bear fruit on a scale commensurate with modern needs, we must invoke the active interest and cooperation of other relevant groups, such as: political scientists, lawyers, economists, sociologists, social psychologists, regional and community planners, educators and publicists. For these latter groups, in a democratic society, are the instruments of social change, the moulders of public opinion, the "engineers" of social innovation and institutional adaptation; their function is to facilitate the adjustment of institutions to techniques with a view toward maximizing the potential gains of scientific progress.

The four preceding papers tend to emphasize and reinforce a conclusion to which I came as a result of my work with the President's Water Resources Policy Commission—and which I had in mind to interject at some appropriate point in the discussion. There I was impressed by the fact that we know so much more than we actually put into practice. I was amazed, and not a little disheartened, to observe that we utilize only a small fraction of the great store of scientific and technical information at our disposal. I asked myself repeatedly why this is so. Why do we fall so far short of our potential level of achievement?

The crux of the matter, it seems to me, lies in the nature of the institutional arrangements—all deeply imbedded in our laws and social attitudes—by which we govern and control the utilization of water resources. These institutions evolved from our pre-scientific, individualistic past; they were appropriate and useful for that period, but we are all aware that those days are gone. We now live in an advanced industrial, urbanized society, and there is a glaring need to modify some of the old institutions which have come down to us from an earlier and simpler past. In short, we need to devise new arrangements consistent with modern needs and modern technology.

Specifically, we should modify those institutions which frustrate the full development and scientific utilization of water resources, and which prevent our scientists and engineers from applying freely their knowledge and creative imagination. The perfection of new institutions is a political problem—here I use politics in the very best sense of the word; it involves a whole series of political actions. For example, I am deeply concerned with our lack of appropriate organizations and financial procedures for the successful execution of large-scale water development projects. Here, we need a great deal of what I call "institutional inventiveness," or creative experimentation.

Consider a few examples: the right of landowners to misuse their lands regardless of adverse social effects, such as accelerated runoff, soil erosion and depletion of natural storage; the right to exhaust groundwaters; the right to dump sewage and industrial wastes into streams and lakes; the doctrine of prior preemption operative in the western states; conflicts between private and public interests; jurisdictional conflicts among local, state and federal agencies, and within the federal bureaucracy; lack of organizations and financial procedures functionally appropriate for large-scale integrated water control systems; lack of rational means for allocation of water to end uses in extreme scarcity situations; lack of adequate governmental power to assess developmental costs in relation to benefits. Such institutional roadblocks and deficiencies must be alleviated before we can attain full application of existing scientific and technical knowledge.

Again, we need to face up to the realities of those relationships we call "rights," to which Mr. Powers referred when he said that industry must be concerned to protect its "rights." In the real world many of these so-called "rights," about which men get so agitated and wrought up emotionally, are either obsolete or nonexistent. When, in Washington, I encountered a gentleman who was greatly

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excited about water "rights" in the West I asked him what they were, beyond mere pieces of paper. He gave me an abstract legal definition that had no relation whatever to reality. The essential fact, in this instance, was that there was no water; the basic scarcity rendered worthless his "right." Yet, when I showed him how, by taking concerted action and by applying scientific principles, an adequate supply of water could be had to validate existing, but now worthless "rights," he took a more realistic attitude and was prepared to accept the institutional changes necessary to get the job done on a scientific basis. Previously he had opposed any such change as tending to jeopardize his "rights."

Another example may be drawn from the area of stream pollution. In the exercise of their ancient rights and privileges, individuals, municipalities and industries discharge untreated waste waters into a natural stream with the consequence that its usefulness is destroyed for all other purposes. Our scientists and engineers know how to clean up this stream, thereby enhancing its value many fold. Yet they are prevented from doing so because we lack the necessary political organization and financial procedures for doing the job on a complete, area-wide basis. Everyone admits we know how to eliminate stream pollution; everyone concedes that pollution of our streams is a great social waste and ought to be prevented; yet, out of concern for individual "rights" and aversion to institutional change, we persist year after year in tolerating this evil.

This is a political problem—and one fraught with extraordinary difficulty because it challenges deep-seated social attitudes and powerful vested interests. But it is not an impossible task. Social attitudes can be modified by the processes of education; vested interests can be compelled to yield to public necessity. It is the special virtue of democracy that its basic processes of free inquiry, free discussion and free legislative action facilitate institutional changes in accordance with the popular will. Scientists and engineers can make a vital and significant contribution to this process by gathering and analyzing the basic data concerning our water resources, by demonstrating improved methods of control and utilization, and by publicizing their findings in meaningful form. Scientific truth will thus provide a solid foundation for intelligent political action designed to raise the general level of efficiency with which we utilize this most basic natural resource.

STATE WATER POLICIES

BY L. R. HOWSON*

WITH DISCUSSIONS BY R. O. JOSLYN, JOHN W. FOSTER, HORACE GRAY, A. M. BUSWELL, AND H. T. CRITCHLOW

BASIC CONSIDERATIONS

In a consideration of water resources policies for the State of Illinois, there are certain basic concepts which are believed to be pertinent:

(1) Illinois is not an arid state. Although it extends more than 350 miles north and south, the precipitation, which averages approximately 35 inches per year, is generally adequate.

(2) The entire State is well watered by rivers and minor streams. The northeastern part of the State borders on Lake Michigan. The entire western boundary is the Mississippi River, and the southern boundary the Ohio River. A large part of the eastern boundary is the Wabash River, and the Illinois River with its 28,000 square miles of drainage area traverses through almost the center of the State.

(3) Underground water, either in the gravels, limestones, or sandstones, is generally available in the northern half of the State. Little ground water is available in the southern half.

(4) In a few areas that were originally provided with abundant underground water resources acute water conditions have been created by substantial overdrafts of both municipal and industrial uses.

(5) Water resources policies in Illinois should be concerned with overabundance, that is, floods as well as inadequacies.

(6) That form of legislative control for Illinois is best which will accomplish the desired results with a minimum of restrictions, administrative procedure and allocations.

(7) State control should be for the common good with enforcement localized to the greatest practicable extent.

It should be pointed out that the State may have a water resources policy without having legislation controlling developments. The primary questions would appear to be, first, has the State an interest in its water resources; and second, to what extent that interest should be expressed in control legislation?

ADEQUATE DATA AND STUDY ESSENTIAL

Adequate data relative to the occurrence and the factors influencing variations in occurrence of water resources and reliable interpretation of those data are essential to proper understanding of water resources whether above or below ground.

With respect to surface waters where the problem of overabundance, as well as inadequacy, is a problem, data relating to rainfall, rainfall intensities, runoff, erosion, silt-carrying characteristics, man-made encroachments in the form of levees and their effect upon flood heights and similar data are important.

Rainfall data, particularly that relating to intensity of fall which is necessary to an adequate interpretation of peak runoff rates, is all too scanty. The problems arising with respect to the storage of water for public use are necessarily studied with inadequate data available on such important matters as probable loss of reservoir capacity due to silting, evaporation from the surface of reservoirs, peak rates of runoff as affecting adequate spillway design, minimum runoff and its duration, etc., all of which are necessary to an adequate consideration of water storage.

With respect to underground water resources the geology and hydrology of the area must be determined before any reasonable adequate plan for the conservation and utilization of our underground water resources can be made. Generally such studies are not made until acute shortages arise and in many cases after heavy financial commitments have been made based upon the assumption that the water resources were adequate for all uses.

Underground water resources are not necessarily all natural. Many underground water supplies have been greatly extended or initially created by artificial recharge. This means of augmenting underground resources has been most largely practiced in the arid sections of the country, but in Peoria, Illinois, there is now in operation, as a research project of the Illinois State Water Survey, an experiment as to the possibilities of recharge to the gravels along the Illinois River on which the draft has materially exceeded the natural recharge.

At Richland, Washington, the site of the Housing Authority for the Hanford Atomic Energy plant, a wholly artificial ground water supply has been created by discharging Yakima and Columbia River water on a pervious gravel mesa from which the water is later withdrawn by wells sunk in the vicinity of the recharge area. A supply of approximately 20 MGD of excellent quality water is being secured

*Alvord, Burdick & Howson, Chicago, Illinois.

in this way at a cost but a fraction of that of purifying Columbia River water through a mechanical rapid sand filtration plant. Des Moines, Iowa, is another example.

It is apparent that the first requirement of any adequately considered water resource policy is a knowledge of the conditions, both surface and underground, as affecting the water resources available and subject to control.

THE LIMITATIONS OF STATE INTEREST

It is believed that the State has an interest in water resources from start to finish, from "dew to slough" or from "cloud bank to river bank." It is believed, for illustration, that this interest includes what has come to be known as rain making. Millions are spent for development and conservation of visible water resources. Why is it not equally pertinent to at least explore the possibilities and, if the possibilities are attractive, eventually the economic practicability of augmenting the visible supplies by those which otherwise are not available? It was a consideration such as this that led the Illinois State Board of Conservation and Natural Resources to match with the Pfister Hybrid Seed Corn Company a $25,000 expenditure to explore the possibilities of rain making.

The same Board recently approved the budget of the State Water Survey, which included an item to finance a study of the use of radar in determining the extent and rates of precipitation over a greater area and with greater accuracy than is practicable with our present sparse network of rain gaging stations, particularly those that determine rates of fall.

It is believed to be a proper function of an organization such as the Illinois State Water Survey to conduct research on water in all its phases insofar as they affect the common good. Its activities should be limited primarily to research pointing out the possibilities and, in general, leaving the determination of economic practicability to those who will benefit therefrom.

It is believed there should be no distinction in the interest of the State as between surface and underground water resources. Underground flow is simply an indication of water on its way to the surface. Today's underground flow may appear as surface flow in the future. Disturbances in underground flow where the aquifers are of broad extent are more gradual in their detectable effect upon the availability of resources than is the case with surface supplies. The storage in the voids of the sands and gravels is usually so extensive as to equalize seasonal excesses or inadequacies and the available resources are, therefore, of a less volatile nature than surface sources. By the same token, overdraft from underground aquifers may be more difficult to correct and the time required for restoration to normal conditions may be much longer than for surface sources.

Since the State is interested in the availability of water resources, it is likewise interested in the maintenance of quality of water which will make it available for all purposes for which it may legitimately be desired. The State, therefore, is vitally interested in pollution control and in many cases pollution abatement.

Pollution control in Illinois is a responsibility of the Sanitary Water Board, which acts in close cooperation with the Illinois State Department of Health. Control over pollution in this State and in its boundary waters, where control is exercised through formal or informal State compacts, may generally be considered as adequate. At least marked progress has been made and with a continuation and improvement in past pollution control procedures, the water resources of Illinois are believed to have all the legislative control necessary from the pollution standpoint.

A State water policy should, of course, exercise such controls as are essentially in the interest of public health. In Illinois, such controls already exist, and are being adequately exercised. The same can generally be stated of all of the 48 states.

LEVEL OF CONTROLS

As a result of the report of the President's Water Resources Policy Commission within the past year entitled, "A Water Policy for the American People," attention has been focused upon national water resources policies. Renewed consideration is being directed toward the level at which water resources policies should be determined and the character and extent of control.

It is believed a sound policy that water resources should be controlled at the lowest level of governmental operations at which such control can be effected. That, it is believed, will require adequate legislation be passed in the several states. In states such as Illinois, it would appear that the legislation should be of the enabling act type by which local communities, in which the overdevelopment or pollution or abuse of natural water resources is of consequence and is adversely affecting the economy of the area, should be permitted to organize a water district within the terms of the state legislative policy. In general, the creation of such a district will probably regulate new and additional water uses, will probably recognize prior developments, and endeavor to apportion the available resources equitably among those needing them.

The State of Illinois by Senate Bill No. 630, signed by Governor Stevenson on August 2, 1951, authorized the incorporation of Water Authorities. This legislation enables the Authorities to inspect

well developments, to require registration of wells, to require permits for additional wells, to regulate use during water shortage, to supplement existing water supply and to levy taxes to accomplish these objectives. This legislation is apparently limited in scope to ground water applications.

The recent legislature also passed Senate Bill No. 292, which is the new water pollution law of Illinois. This act gives the Sanitary Water Board broader powers over pollution abatement.

It should be pointed out that the regulation of many so-called water shortages in a state such as Illinois, which has abundant water resources, in many localities a choice of resources is quite largely an economic question. Most sections of the State have some choice of supplies, the one selected usually being the lowest in annual cost. As time goes on, the economic comparison between supplies changes and the use is frequently transferred to another source. An illustration of this, of course, is the Chicago area, which is underlaid with sandstones which have their outcrops in central Wisconsin. Some 50 years ago, wells penetrating these sandstones in the Chicago area flowed freely at the ground surface. While the City of Chicago drew its water supply from Lake Michigan, practically all of the suburbs and many of the major industries drew their supplies from wells penetrating the sandstone. As the draft increased, the water level receded until, at the present time, the water level in wells penetrating the sandstones is 350 feet or more below ground level in practically all of the Chicago area. As the water level receded, the major users found it more economical to connect to the City of Chicago, and progressively such communities as Cicero, Berwyn, Oak Park, Brookfield, the Clearing Industrial District, the Corn Products Company, all large users of water, and others gave up their wells. With each reduction in use, there would be a retardation in drop or a recovery in well water levels. Thus economics, rather than legal requirements, have largely dictated the choice and regulated the source of water supply in the Chicago area.

In this connection a statement which appears in the Engineering Joint Council Committee Report entitled, "A Statement of Desirable Policy with Respect to the Conservation, Development, and Use of the National Water Resources," is pertinent. It applies to the question Mr. Alquist brought out. This statement reads:

"The Coordinating Committee points out in connection with public water supplies that, contrary to general public belief, there is no experience basis for the fear that the availability of water resources, surface or underground, is declining. Shortage of water is neither universal nor increasing. Synchronization of development and of use on occasion has been inadequate. Declining underground water levels prevail in some areas, with excessive ground water rises in others. On a national scale, however, there is little evidence to support the prophets of doom in the water resources field. Intelligent husbanding and allocation are the keys to the future."

FINANCING

In general, a water resources policy should recognize that those who benefit should pay the cost and that benefits should materially exceed costs on work that is done with public funds.

An illustration—the State of Illinois built and paid for the Illinois Waterway which has now become an important link in our national inland waterways transportation system, 20 million tons a year. As such, it is believed proper that it should now be operated and maintained by the Federal Corps of Engineers, which it is.

Since, other than navigation, water resources development in the State of Illinois is largely local, essentially intrastate and, to a minor extent only, interstate which could be handled by state compacts, the Federal Government's financial participation should be, I believe, limited to supporting such federal agencies as the U. S. Public Health Service in its research and education work, in cooperating with the State, establishing and operating stream gaging stations and similar acts for the common good, including navigation. Water resources, to repeat, should be developed and paid for by those who benefit therefrom.

Considered with respect to the next lower level, having discussed Federal, it is believed that State participation in water resources should be confined to research and that those who will profit from each individual development, again, should pay its cost.

Proceeding along the above lines of reasoning, the Illinois State Water Survey, as previously mentioned, has conducted research on the basic problems of rain making, on meteorology as related to areas and intensities of rainfall, on reservoir storage projects for public water supply to secure data on flood runoff as affecting spillway capacity, silting, evaporation from exposed water surfaces, etc. The Water Survey has also performed valuable service in calling attention to the cost of hard water, the factors affecting corrosion, research on the factors affecting the rapid deterioration of deep well turbine pumping equipment, conducted some of the early research on the activated sludge process of sewage treatment, on the efficiency of trickling filters at various depths of the stone bed, factors affecting digestion and fermentation of sewage and industrial waste solids and many other research problems which have been developed and utilized with large economic savings both within and without the State of Illinois.

While recognizing the important accomplishments of the Illinois State Water Survey with respect to the development and preservation of our water resources and taking pride in them, the work remaining to be done is a challenge which the Illinois State Water Survey accepts in its definition of value as being a measure of future usefulness.

DISCUSSION

RAY O. JOSLYN.*—Mr. Howson has very ably presented a comprehensive paper on State Water Policies and limited the subject to the State of Illinois.

Mr. Howson has mentioned the relationship and obligation of the Illinois Water Survey to various groups, and in this discussion I wish to enlarge upon that phase of the subject. The three points which I shall discuss are the relationship of the Illinois Water Survey

1. With the Public.
2. With the Federal Government.
3. With Private Business.

RELATIONSHIP OF THE ILLINOIS WATER SURVEY WITH THE PUBLIC

The Illinois Water Survey naturally owes its first obligation to the citizens and taxpayers of the State of Illinois. In fact, its continued existence depends upon this group. How then can it best serve the common good? The water resources of the State of Illinois are one of its greatest assets and like any product, a complete knowledge of its availability and application greatly enhances its value. Therefore, it is a basic function of the Survey to collect all available technical information on water supplies—both surface and underground; to correlate the data from test borings as an aid in establishing underground valleys and reservoirs; to maintain records of producing well capacities and drawdowns so the main water-bearing strata can be classified; to obtain logs of existing and future wells and to assemble data on rainfall, runoff and other pertinent subjects. The Water Survey should be a technical library of information of the water resources of the State. This information should be made conveniently available in understandable language to city officials, engineers, contractors, and the public in general.

As a further service the Survey should expand their investigations and should serve as a research institute in obtaining information on water flows in streams, runoff, permeability and transmissibility of water sands, rehabilitation of wells, recharge of formations with surface water or discharge wells, corrosion of pumps and casings and new methods of equipment to better understand the water resources of the State. How will the above serve the taxpayer of Illinois?

(1) Industries will be attracted to the State, knowing that accurate and dependable water information is available.

(2) Municipalities can secure information regarding new and supplemental water supplies.

(3) Consulting engineers and contractors can obtain data to aid them in reports and proposals in their work and as a result of the above.

(4) The public will benefit in being able to obtain more dependable water supplies, in both quality and quantity.

The relationship to the public is to provide a technical library and a research institute.

RELATIONSHIP OF THE ILLINOIS WATER SURVEY WITH THE FEDERAL GOVERNMENT

Mr. Howson has stated that the Federal Government's financial participation should be limited to supporting such federal agencies as the U. S. Public Health Service in its research and educational work, in cooperating in mapping the state, establishing and operating stream gaging stations and similar acts for the common good, including navigation. To this plan I wholeheartedly subscribe; let us see what happens when the federal participation exceeds the limitations stipulated above.

With the addition of contributed federal funds, the services will be expanded and additional data secured, but in the zeal of expansion the State Survey may find it difficult to limit their activity to the purpose for which it was originally created.

We find that in some of the states which have received federal funds on a cooperative basis, the State Surveys have been urged and requested to purchase test drilling equipment for the drilling of test holes, which test holes were drilled primarily to secure regional geological information. However, many of the test holes were drilled in locations that could be classed as regional or local and in some cases tests were drilled for a local municipality. Whether this was due to local pressure, or political persuasion, is inconsequential but if means that the Survey was ignoring and losing sight of its main objective of securing regional data.

Municipalities are prone to ask too much of state and federal agencies and, wanting to get "something for nothing," they are inclined to request engineering service which should be provided at community expense. They must be repeatedly reminded that the tax-supported agency has for its main objective the securing of data to complete the state-wide picture.

Aggressive engineers in public employ have been known to exceed their function by conducting test drilling programs for cities or by the application of data to the detailed solution of local problems. These are clearly practices to be deplored.

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They invade the province of private industry (the consulting engineer and the drilling contractor); they result in favoritism since such service cannot be offered to all who desire it; and they divert personnel away from much-needed basic studies. Many other direct or indirect objections to such "socialized engineering" are at once apparent.

This policy of donating service to a city is in conflict with the ideas expressed in Mr. Howson's paper, wherein he states, "that State participation in water resources should be confined to research, and that those who will profit from each individual development should pay its cost."

Therefore, the relationship between the State Survey and the Federal Government should be one of cooperation in exchanging data and information, but certainly a state should be sufficiently interested in its own resources and financially able to lend all monetary assistance necessary to support its own State Water Survey, and the state should not request or accept federal funds to assist in carrying on this work.

It is quite noteworthy that among the several states with which the writer has frequent contact in regard to water information, the States of Missouri and Illinois are outstanding in having authentic information readily available and neither of these States receives federal appropriations on a cooperative basis.

RELATIONSHIP WITH PRIVATE BUSINESS

What type of relationship should exist between the State Survey and Private Business? Certainly it should be one of harmony, cooperation and respect and in no way should Private Business ever have any reason to believe that the State Survey is competing with Private Business. The private driller and engineer should be anxious to turn over to the State Survey all information on logs of wells, pumping data, and interference of wells so that this information can be tabulated and used to augment existing data. The individual who is now contributing to the over-all fund of knowledge will be able to refer to this and other data in the future.

Due to complaints by Private Industry of the interference of the Surveys, mentioned a few moments ago, a conference was held in May of 1948 in Atlantic City, with the policy committee of the American Water Works Association, representatives of the U. S. G. S. and representatives of Private Business. An official statement of A. W. W. A. policy committee as a result of that meeting is quoted below:

"The Association holds the opinion that neither the U. S. G S. nor the State agencies with which it cooperates, should own or operate test well drilling equipment or equipment suitable for drilling water wells for later use. On the contrary, the Association holds the opinion that test well drilling should be done by competent private industry under contractual arrangements made by the driller with the U. S. G. S. or the State or local public agency concerned. In outlining these opinions in broad principle, the Association grants that when test well drilling needs to be done—and no private contractor is willing or able to do the work—the public agency may properly do it in order that the public interest be served effectively."

In view of the above, it was interesting to note a recent press report which reads as follows: "The Chairman of the City Council water committee, made contact with the head of the Geological Department and secured assurance that his department would make a search for good well water for the City. The geological department has a special service available which will make a careful survey of the sub-terranean well water possibilities in this area. The University men plan to begin work within the next week or ten days."

Naturally, it is not a pleasant sight to a private well contractor to see a state-owned well rig drilling test holes while his rig is standing idly by. If he were one of the fortunate well drillers who made a profit in his business and paid taxes, part of his tax money was used to buy the rig which in turn was used to compete with him.

Certainly, no individual or company in our form of government should have to compete with his own government, either state or federal, in performing his particular line of business. Therefore, in summarizing this point, it should be stated that there should be no semblance of competition between the Survey and Private Business; on the contrary, with a program of cooperation, harmony and intelligent foresight, the Survey and Business should march forward together, arm in arm, to give the people of this great state full knowledge of their greatest heritage—WATER RESOURCES.

* * * * *

JOHN W. FOSTER.*—As a geologist may I comment on a few points which have arisen here this morning? We think of ground water as a renewable geologic resource, and therefore its efficient development requires the coordination of geology and engineering.

The State Geological Survey cooperates very closely with the State Water Survey in research on our State's water resources. The State Geological Survey is of the opinion that we have discovered and utilized only a small fraction of the total ground water resources available in the State. We have tremendous supplies of ground water that are just beginning to be tapped. Illinois is fortunate to have pre-glacial valleys, now to a large extent completely buried. Many of the glacial gravels contain untouched water resources. Much of Illinois remains to be explored by systematic methods.

The Illinois State Geological Survey does work in exploration but we have no drilling rigs in operation, and I am sure Mr. Joslyn's remarks on that were not directed at Illinois activities. We do use an electrical resistivity method for locating areas suitable for test drilling. We have helped a great many communities in Illinois in locating deposits suitable for the community water supply. These communities might otherwise have gone without development of their water resources, being unable to afford exploration by random test drilling. Much of our work involves the mapping of areas which appear barren and unworthy of test drilling. We often point out areas favorable for test drilling, but it is not our practice to point out specific sites for well location. I would like to stress again the need for geologic and engineering coordination and that the State Geological Survey works very closely with the State Water Survey in research on the ground water resources in Illinois.

R. O. JOSLYN.—I think, Mr. Foster, that, at present, private industry is not able to carry on resistivity exploration as effectively as the State. When this method does become common in private industry, then private industry can do it.

HORACE GRAY.—Mr. Joslyn has had an opportunity to deal with the organization of water authorities in various states. It occurred to me to ask him whether he regarded Illinois' new Water Authority bill as adequate to do the job on a community or district basis.

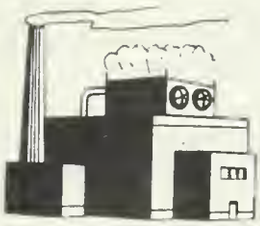
R. O. JOSLYN.—It is a very brief bill, and generally it is typically adapted to Illinois conditions. It is, I think, applicable only to underground conditions. I am not sure that this is our whole problem. I believe the thinking might be expanded to better control of some surface supplies too, but it is a creative-type of bill, adaptable to local conditions. Control should be left to local communities where there is occasion for regular organization and regulation by themselves.

A. M. BUSWELL.—I think the bill was drawn essentially to cover a situation in Peoria. It is my impression that it may be extended to surface water if the conditions require. The bill makes it possible to handle water use on the lowest political level. It makes it possible for the smallest area to handle its own problem. I have an example of the advantage of local control. We have had a condition of recession in the Champaign-Urbana community, and it has been necessary to go some distance to supplement our water supply. There were two local industries with a mutual problem; a soybean plant required a large amount of water for cooling, and a railroad required water which was warm. The soybean plant was not successful in finding a well with a large yield, and the railroad had a substantial ground water supply. The Water Survey suggested that the railroad could rent the water to the soybean mill and then use it for its own purposes afterwards. The necessary pipelines have been laid and put into use so that sort of free cooperation is operating here. Many of us, if we will take the trouble to go into the details and contact the individuals, can solve similar water problems in a smaller area than a municipality.

H. T. CRITCHLOW.--I was interested in Mr. Howson's comments from the standpoint of having the control over surface water for public and private use and for underground waters. The experience in administering such a law in New Jersey developed a conflict between the users of water for public supply and the private users of well water in that the public supply had no protection against the encroachment upon their sources by the private wells. This situation finally resulted in the study and passage of a law which gives the State authority to delineate and place under regulation certain areas within the State to protect the sources of supply from undue pumpage, or encroachment of salt water. Those areas have been delineated where the need seems to dictate the necessity. That law has been on the statute book for four years, and it has been rather surprising to me that the industrialists and the other owners of private wells welcome some control because the law does provide that those wells, or the well owners, have established certain rights. This was a natural provision in the law itself. So in the administration of the law, while we have a great many difficult problems to solve, everyone seems to welcome the control by the State in these areas which are called protected areas. Our experience there would indicate the practicability of having permissive regulation which would limit the control to the areas where it is needed to protect supplies from damage by pollution or overpumpage.

*Assistant Geologist, Illinois State Geological Survey, Urbana, Illinois.

T R E A T M E N T



Program Chairman

T. E. LARSON

Treatment

Room 116, East Chemistry Building, southeast corner, Mathews Street and California Avenue, Urbana.

MONDAY, OCTOBER 1

Evening

6:30 Informal get-together, Urbana-Lincoln Hotel
Buffet supper. Football and radar movies.

TUESDAY, OCTOBER 2

Morning Session
C. H. Spaulding, Presiding

9:00 "Address of Welcome," Roger Adams
9:10 "Corrosion Fundamentals," H. H. Uhlig
10:10 Discussion

10:45 "Composition of Water Substance," W. H. Rodebush
11:45 Discussion

Afternoon Session

1:45 "Exchange Materials," A. S. Behrman
2:45 Discussion

3:15 "Boiler Feedwater Conditioning," F. G. Straub
4:15 Discussion

* * *

5:30 Reception, Illini Union Building

6:30 Banquet, Illini Union Building

SURVEY OF CORROSION CONTROL IN WATER SYSTEMS*

BY HERBERT H. UHLIG**

WITH DISCUSSIONS BY E. N. ALQUIST, JOHN F. WILKES, CHAS. H. SPAULDING, R. C. BARDWELL, H. H. UHLIG

Losses through corrosion of metals probably first received attention at the turn of this century, when the utilization of iron and steel took a sharp turn upwards. The estimated figures for these losses had a certain interest as statistics, but it often remained easier, practically, to replace corroded structures than to trace the source of deterioration. Today the emphasis has changed, and the need for corrosion control is accented particularly as metal supplies become short, as ore reserves look emaciated, and as modern industrial equipment must be better protected against increased operating pressures, temperatures and other corrosive factors.

Certainly the economics of the situation leaves no doubt that we should know more about corrosion than we do. Replacement costs for corroded equipment have become more rather than less disturbing, and shut downs, contamination of products, loss of efficiency, and accidents have never been popular. Hiding of ignorance by ample overdesign to take care of the unknown in corrosion, is less acceptable engineering practice today than it was in times past. The pipe line, the pump, or the oil well sucker rod must now entail the optimum economic design meeting both mechanical specifications and expected corrosion over a long range basis. The acceptable approach to corrosion problems, in other words, has become more subtle than merely increasing the cross sectional area of metal. Cathodic protection, metal coatings, organic coatings, inhibitors, treatment of the environment and alloys offer proved advantages in the saving of metals, human effort and dollars. Much has been done in recent years to mitigate corrosion, but across the board in all industries and wherever metals are used, the major advances along these lines are still to come.

What we presently know about corrosion mitigation has accumulated through several sources, including (a) basic research and (b) service data combined with empirical probing. Stainless steels, for example, were discovered in England by the empirical probing of H. Brearley in 1912. The use of modern protective paints and chromate inhibitors also resulted from probing combined with service data. On the other hand, modern boiler water treatment, the application of the Saturation Index to corrosion control, the discovery of cathodic protection, and the improved corrosion resistance of present-day aluminum alloys were the result of planned experiments and fundamental knowledge of the sciences.

Fundamental research in corrosion began in the early 1800's with the basic work in electrochemistry of Sir Humphrey Davy, Michael Faraday and W. H. Wollaston in England; A. Volta in Italy; and A. de La Rive in France (1)[. The quantitative relation between electricity and chemical change in a cell was firmly established; from this the suggestion followed that corrosion of metals must be similar to the action that takes place in the galvanic cell. Cathodic protection, one of the most effective, practical present-day means for reducing corrosion of metals, had its beginnings at this time in the systematic research of Davy. His laboratory experiments showed that zinc, tin or iron diminished corrosion of copper to which it was electrically coupled in a salt solution, an idea which he applied later to protect the copper sheathing of a British warship.

Although this early work established the foundations, it was not until 75 years or so later, at the turn of the 20th century, that the subject received further attention. By this time, the industrial age was well under way with its tremendous demands on metals, and corrosion began to make an impression as a subject of technical and economic importance. Willis R. Whitney in 1903 (2), while a member of the teaching staff of Massachusetts Institute of Technology, focused attention on the electrochemical processes attending the corrosion of iron, followed a few years later by the researches of his colleague, W. H. Walker (3). Walker and his collaborators proved the important role of dissolved oxygen in the corrosion of iron, and showed that carbonic acid was not necessary to the reaction, contrary to the general impression up to that time. From this was built the foundations of modern boiler water treatment, making possible the economic

*Also submitted to the American Chemical Society for publication in conjunction with Symposium on "Corrosion by Water" at 75th Meeting, September 4, 1951.

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[Number in parentheses refers to number in BIBLIOGRAPHY at end of this article.

production of high-pressure steam in boilers constructed of steel, from which followed, in turn, cheap electrical power. Corrosion control was an essential step in this development.

Basic research gained impetus in the 1920's, and included the investigations of W. G. Whitman and those associated with him on the behavior of iron and steel in waters as a function of pH (4). This work provided a fundamental answer to the age-old controversy of whether ancient iron was better than modern, or some brand of steel or special kind of iron was better than its competitor, a question which service data had seemingly left open. Within the range of pH of most natural waters, oxygen depolarization controlled the rate of corrosion, and oxygen concentration was therefore the important factor. The gross composition, or incidental alloying elements, or inclusions, consequently had little, if any, effect on the rate. When this plausible conclusion was announced, a vast amount of service data in fresh waters and in sea water, either then available or obtained later, confirmed the laboratory deductions. It was this research which also explained why heat treatment, internal stresses and surface preparation of a steel, despite erroneous intuitive concepts, were found to have practically no effect on the corrosion rate in natural waters.

In the acid region, on the other hand (below pH 4 to 6 depending on the nature of the negative ions), where hydrogen evolution is the major cathodic reaction, it was demonstrated that composition of iron and steel becomes important. The controlling reaction at the minute cathodes of the steel surface in this region of pH, is determined not so much by oxygen concentration, as by hydrogen overvoltage. For this reason, surface finish, inclusions, alloying elements, and presence or absence of stress play a part. Therefore, although any iron or steel may be equally suited to fresh or salt water,* some attention to composition is worthwhile when specifications are set up for a steam-return line handling hot carbonic acid or for similar installations.

These are only a few instances showing the relation between basic research in corrosion, which began 150 years ago abroad and 50 years ago in this country, to modern industrial development and to a high standard of living. They introduce the next section of this paper because throughout the development of methods for combating corrosion, fundamental research has played an indispensable role. And more important, it is perhaps to fundamental research we must look almost entirely for major advances in the future. In all fields of technical development, when the obvious trial and error methods reach diminishing returns, only systematic and unprejudiced scientific search for further truth offers real hope of progress. Advances in corrosion control are no exception.

*Cast iron corrodes by so-called graphitic corrosion, whereby the iron constituent of the alloy is converted to corrosion products, these acting as a cement for residual graphite flakes. A corroded cast-iron pipe, therefore, may have lost most of its mechanical strength, but is still serviceable for the purpose first intended. In this respect, it may last longer in some applications than steel or iron. It is also reported that a 3% Cr steel, even though its weight loss is parallel with other steels, is less subject to penetration by pitting.

SURVEY OF CORROSION MITIGATION

A survey of present means for corrosion mitigation discloses obvious gaps in the present state of the art or science, as the case may be. Until these gaps are reasonably filled, the protection of metals from corrosion will remain incomplete, and the annual waste (5) from this source will continue to be large. The current large-scale application of cathodic protection has offered the most recent evidence that some of the savings and conservation so much needed are being accomplished.

CATHODIC PROTECTION

Cathodic protection is now applied largely for protecting buried pipe lines transporting oil, gas and water, and, to a lesser extent, for protecting structures exposed to natural waters, such as canal gates and industrial water tanks. By passing a current through the water or soil in the order of 0.0001 to 0.1 amp./sq. ft. of structure surface, the actual value varying with the nature of the environment, minute galvanic currents on the surface of the metal accounting for corrosion are neutralized, whereupon reaction ceases. The impressed current for cathodic protection is supplied from a rectifier or generator, or may be produced by the chemical energy of galvanic anodes such as magnesium. The life of structures cathodically protected is prolonged almost indefinitely at the modest cost of installation and electricity, or of the sacrificial metal used for galvanic anodes, and is potentially useful in many places where metals are used. Cathodic protection for this reason eventually will be applied more widely, and will undoubtedly enter as a factor in prolonging the life of metals used in water and sewage treatment.

Galvanized hot-water tanks are now being protected by magnesium anodes contained in the tank, thereby adding several years to their life. Industrial water tanks are frequently protected using an impressed rectified A-C current and corrosion resistant anodes. The Panama Canal gates, covered with a hot bituminous coating, are protected using an impressed current of 0.001 amp./sq. ft. and replaceable steel anodes (6).

The method is also applicable for protecting the

outside of pipes, buried or totally immersed, whatever the corrosive medium in contact. Perhaps it is worth mentioning that when so applied it has no effect on corrosion of the inside surface. In order to protect the inside, it would be necessary to insert a conforming anode reaching the entire length of the pipe, which obviously is not easily done. In addition, reaction products at the anode or of the anode itself are retained within the pipe, and must therefore exert no undesirable effects on the water or water solutions carried by the pipe system.

It is present practice to apply an organic coating conjointly with cathodic protection, thereby reducing the required current for protection, and better distributing it. Since the coating is insulating, the current flows only to breaks or pinholes in the coating, which are precisely the areas where corrosion occurs. One magnesium anode buried in the ground can often protect five miles of a coated buried pipe line, but would be effective in protecting perhaps only 10 to 100 feet of uncoated pipe. Because alkalies are one of the reaction products at the cathode according to the reaction,

$$2H_2O \longrightarrow H_2 + 2OH^- - 2\,e, \text{ or}$$

$$1/2\ O_2 + H_2O \longrightarrow 2OH^- - 2e$$

the organic coating must be alkali resistant.

Cathodic protection is not effective above the soil or water line, and cannot be applied to alleviate atmospheric or high temperature attack. It can be applied to metals other than iron and steel, with the one precaution that amphoteric metals like aluminum, zinc or lead, which are attacked by alkalies, must not be overprotected by use of high currents, since the accumulated alkali will corrode the metal. With iron or steel, overprotection is not critical, because alkalies act as corrosion inhibitors.

It is not often appreciated that cathodic protection, in addition to being useful for protecting metals against rust or general surface attack, also prevents dezincification, pitting, intergranular corrosion, corrosion fatigue (that portion of fatigue accelerated by corrosion) and stress corrosion cracking. Stainless steels, for example, which invariably pit upon exposure to sea water, are immune when cathodically protected. Likewise, brass which dezincifies or which may season crack (stress corrosion crack) when stressed in certain environments can be protected by this means. The possible applications of cathodic protection in specific instances of this kind are legion and have only begun to be appreciated.

However, the basic science of this approach to corrosion control trails behind practice. Urgently needed today are quantitative data relating corrosive factors of the environment, such as pH, velocity, dissolved salts, galvanic couples, surface finish and temperature to the minimum required current for total protection. Also needed are fundamentally established criteria of complete protection more satisfactory than those now in use. Overprotection, with its added costs, or damage in some instances, would then be avoided. It is reasonable to expect improvement in the efficiency (reduction of local corrosion) of sacrificial anodes above the present 50% figure, and the development of more efficient insoluble anodes, having lower oxygen overvoltage, in applications where impressed currents are applied. These are but a few of the many problems.

METALLIC COATINGS

The metallic coatings used in largest quantity are tin, zinc (galvanized) and nickel coatings on steel. Tin coatings are employed largely for food and other containers (tin cans) for which they are singularly well suited from the standpoint of corrosion protection and lack of toxicity. Zinc coatings are relatively effective in avoiding rust of iron whether exposed to the atmosphere, buried in the soil, or immersed in water. The life of the coating increases with coating thickness. The principle of protection is the same as that explained under cathodic protection, the zinc corroding sacrificially and the resulting current protecting iron cathodically. Only in some hot waters above approximately 140° F. does zinc fail to protect iron, a fact only recently established by laboratory experiments (7), and confirmed by service data (8) and further laboratory tests (9). This is another illustration of how fundamental work in this field points the way, whereas service tests requiring so long a time before evaluation and involving so many uncontrolled variables during the test often fall short of being conclusive.

Nickel coatings are noble to iron in the galvanic series, and must be relatively pore-free in order to protect the base metal from attack. This is accomplished by plating a sufficient thickness of metal, usually in the order of 0.0005 to 0.0015 inch. Presumably, much better protection will eventually be achieved for the same thickness of metal when commercial plating practice makes it possible to electrodeposit nickel with fewer numbers of pores. This is a reasonable objective of present-day plating research, and is receiving some attention.

Clad coatings, such as stainless steel or nickel, bonded metallurgically to steel, or pure aluminum bonded to a stronger aluminum alloy, are free of pores, and offer protection equivalent to the bulk metal constituting the overlay. Some design problems are encountered in handling edges of such composite materials where the underlying metal is exposed, but, in general, the problems can be handled satisfactorily. The costs run high com-

pared with electrodeposited coatings.

Titanium-clad steel may be the ultimate answer to many applications involving exposure to salt solutions or hot waters, particularly as the price of titanium is lowered. Unlike 18-8 stainless steel, this metal has been resistant to crevice corrosion and to pitting in sea water in all tests conducted so far.

Further developments in metal coatings may eventually provide (a) new types of sacrificial coatings in the form of alloys, (b) reduced porosity of noble-metal coatings, (c) cladding processes less expensive than those now available, and (d) sprayed metal coatings with satisfactory impermeability and lower in cost than present coatings of this kind. Sprayed coatings have the inherent advantage that they can be applied to finished structures.

ORGANIC COATINGS

Compared with their good performance in atmospheric exposures, the common drying-oil paints have been only fair when applied to metal structures totally immersed in natural waters or buried underground. Their life is short even under ideal conditions. Multiple coatings, baked on, of some synthetic resin paints are better but expensive, and it is seldom possible to guarantee that all edges or corners are coated adequately, or will remain so during service.

Phosphating of surfaces before painting has usually proved beneficial in improving protective qualities of all paints. Heavy bituminous coatings remain the standard for buried pipes, and are also being used on the inside of water mains. Heavy plastic coatings and rubber, bonded to steel, offer good protection to severely corrosive chemical environments where they are used with economic advantage. One of the most chemically resistant of modern plastics is the tetrafluoroethylene type, which resists boiling acids and alkalies, all solvents, and corrosive gases like chlorine. It is so successfully resistant that bonding it to metals has presented a major problem. It is a reasonable expectation that materials of this kind will eventually enjoy wide application in many industries. Perhaps their superior resistance will eventually be incorporated into paints having protective qualities marking an improvement over present paints by several orders of magnitude.

Among the inexpensive coatings, portland cement continues to offer outstanding protection to iron and steel against hot or cold waters and to the soil. Water pipes in service for 60 years or more are not uncommon. Glass-covered or enameled steel surfaces also offer good protection, but are easily damaged mechanically or when subject to thermal shock, making them applicable largely to specific chemical environments, or to atmospheric exposure.

INHIBITORS

Chromates and nitrites continue to be the most efficient of our present-day inhibitors for controlling corrosion of iron and steel. They suffer by being toxic, and, hence, are not applicable where potable waters or food are handled. They also have limited use at elevated temperatures, or where salt concentrations are appreciable. Sodium silicate continues in use for treating potable soft waters, effectively reducing pick up of flocculent rust, as well as reducing dezincification of brass. It has not been outstandingly successful in preventing attack of galvanized domestic hot-water tanks.

Glassy metaphosphates are being used in a few parts per million to combat corrosion of steel in non-recirculating cold-water systems. Recent work in the Corrosion Laboratory at M. I. T. indicates that dissolved oxygen is beneficial to the efficiency of metaphosphates in a degree not previously suspected, although the effect has been known (10). The mechanism of behavior of inhibitors such as these, and a better understanding of their proper application is one of the present projects at M. I. T. supported by the Office of Naval Research.

Orthophosphates are widely used as corrosion inhibitors in boilers, both for minimizing localized attack of tubes and avoiding stress corrosion cracking. Inhibitors such as sodium nitrate and quebracho extract also find use in avoiding stress corrosion cracking (caustic embrittlement) of boilers.

Basic comparative data on all commercially available inhibitors are needed today relating their efficiency to temperature, pH, dissolved chlorides, sulfates, nitrates, calcium and magnesium, dissolved oxygen and organic materials. The disturbing effect of galvanic couples also requires adequate evaluation. There seems to be no doubt that a good, nontoxic, inexpensive inhibitor for potable waters, particularly if it were effective in hot-water systems, is presently needed and would be welcomed by the water industry.

ALTERATION OF ENVIRONMENT

Deaeration of boiler waters probably constitutes the most important example of mitigating corrosion of expensive equipment by altering the environment. Oxygen removal is also finding use to reduce corrosion of pipes and steel equipment handling cold and hot waters, where the required oxygen content need be only 0. 1 to 0. 3 ppm., instead of virtually zero as in high-pressure boilers. This approach may eventually prove worthwhile for corrosion control of municipal water systems, since it also solves simultaneously the householder's problem with respect to piping and hot-water tanks. The normal tests of the water may need to be restored by an aerator at cold-water faucets. One problem may

possibly arise in this connection, particularly in applying vacuum deaeration, through the enhanced opportunity for sulfate-reducing bacteria to accelerate corrosion. These bacteria thrive only in waters of low-oxygen content, and corrode ferrous metals many times more rapidly than normally aerated water. Their control, in some instances, has been accomplished by means such as chlorination.

Deaeration of waters will also reduce corrosion of copper, brass and lead, but not necessarily stainless steels or aluminum, the latter depending on dissolved oxygen for their passivity or optimum corrosion resistance to aqueous media.

On this same subject, Groesbeck and Waldron (11) showed that increased dissolved oxygen, although at first stimulating attack, reduces the corrosion rate of iron in distilled water above concentrations of 16 ml./liter. This result has been confirmed by others. The practical value of these data has not yet been assayed in large-scale equipment, but the suggestion occurs that oxygenation of soft waters may be one possible way to reduce corrosivity of such waters in contact with steel. The mechanism proposed for this behavior is one either of building up a more protective barrier iron-oxide film, or chemisorption of an oxygen film on the iron equivalent to a monolayer or less (12). Reduction of corrosion by oxygenation is not expected for copper or brass, nor for waters containing appreciable dissolved salts, such as chlorides and sulfates, or at high temperatures, because the films responsible for passivity either no longer form on iron, or are less protective.

It was known for many years that some natural fresh waters are more corrosive than others. The difference eventually resolved itself into the presence or absence of a protective layer on the surface of the metal, usually calcium carbonate. It remained for Langelier in 1936 (13) to analyze this problem quantitatively from the standpoint of physical chemistry, and to propose as a result the Langelier or Saturation Index. Waters with positive Index are supersaturated with respect to calcium carbonate; hence, they can deposit a protective film and are nonaggressive from the standpoint of corrosion. On the other hand, waters with negative Index are undersaturated and aggressive. This convenient classification of waters has led to a scientific basis for corrosion control of water systems and has proved useful in many instances. Only where conditions are such that calcium carbonate cannot precipitate as a continuous protective film, as when colloids are present, does the rule break down. Also, occasionally heavy scaling occurs in waters of positive Index at elevated temperatures. Further work along these lines will undoubtedly take care of the special situations.

METALS AND ALLOYS

Galvanized iron remains the standard material for handling cold and hot waters of positive Saturation Index. In some hot waters, galvanized iron may pit more than ungalvanized iron, as mentioned previously. For waters of negative Index, copper and red brass for small-size pipe offer economic advantages over steel, whether galvanized or not. Monel (70% Ni - 30% Cu) and copper are useful for hot-water tanks in soft water areas. Aluminum is satisfactory for distilled water, but is not satisfactorily corrosion-resistant to all types of waters, especially when the waters have been previously in contact with iron or copper. Magnesium, likewise, is sensitive to impurities in the water, and, as yet, cannot compete economically with the heavier metals mentioned above. Lead is presently expensive and, although durable, may contaminate soft waters with lead salts sufficient to render the water unsafe for drinking. This fact should be borne in mind when water softeners are installed in some of the older cities, where lead piping is still in use. Stainless steels are not used widely because of expense, and also because they may pit rapidly in some waters containing dissolved chlorides, especially when ferric and cupric ions are present simultaneously. Titanium holds promise of application in the handling of hot and cold waters, if and when the price approaches that of the stainless steels.

SUMMARY AND CONCLUSIONS

Corrosion of metals used for waterworks and boiler operations will continue to present problems for a long time to come, but the over-all losses from this source will decrease as basic research in corrosion gains support, and workers are attracted to the challenges of a field in which any degree of success may amount to millions of dollars saved. A reliable, inexpensive, nontoxic inhibitor is needed to help reduce tuberculation and clogging of water pipes, as well as to protect hot-water tanks. Failing this, deaeration of waters at the source offers some hope and promise. Intentional oxygenation of cold, soft waters, which are the most corrosive type, may also prove useful in special cases. Lime and sodium carbonate additions, in accord with Langelier's development, continue to be a useful approach to corrosion control.

Cathodic protection should find many economic applications in controlling corrosion of waterworks equipment, and will probably be applied more generally the next five or ten years. Improvement of corrosion protective paints and organic coatings, both from standpoint of performance and price, should be accomplished in the near future, and will

help further reduce the expense and burden of metal deterioration. Of metals available to resist attack marking an improvement over those now used, titanium offers the most promise. When the price approaches that of the stainless steels, it will find use as sheet and as a clad coating over steel.

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DISCUSSION

E. N. ALQUIST.[1]—I would like to ask, Professor Uhlig, regarding the Langelier formula where you lay down this calcium carbonate scale. During the War we tried that with assistance from a number of reserve technical people in two operations: one in California and one in Texas (as the last one), and our experience was not very good with that. I want to inquire if there has been set up somewhere in the basic research picture, work along that line to iron out these difficulties that we encountered and others have encountered; and I would like to know if there is anyone in the audience who has had experience with it that is good and increase my own personal knowledge of the subject.

PROFESSOR UHLIG. —To answer your last question first, there is ample evidence to prove that waters depositing a calcium carbonate scale (hence of positive Saturation Index) are less corrosive than soft waters (negative Saturation Index). For example, over many years of observation, Chicago water has been consistently less corrosive to iron or steel than soft Boston water, the essential difference being due to the relative ability of Chicago water to lay down a protective layer of calcium carbonate. This difference in corrosivities of waters was made use of in formulating the Saturation Index.

Soft waters are treated with just sufficient lime or soda ash to bring about slight supersaturation of calcium carbonate, thereby assuring deposition of a protective calcium carbonate scale. The method fails to meet expectations only when conditions are such that the calcium carbonate is precipitated in a noncontinuous, nonprotective condition. This occurs, for example, when certain colloids are present in the water which serve as nuclei for growth of calcium carbonate crystals in the body of the liquid. Also, protection may be inadequate when certain soluble salts are present, the exact nature and critical concentration of which are not known so well as we should like. Further studies are needed. But in any case, lime or alkali treatment, in accord with the Saturation Index development, is a useful means of corrosion mitigation. However, the waters to which it may possibly apply should be examined in more detail than by chemical analyses for calcium, pH and alkalinity alone.

* * *

[1]Dow Chemical Co., Midland, Michigan.

UNIDENTIFIED.*—I am very much interested in your statement that soft waters might be amenable to treatment by overaeration. Are there any practical applications of that? In the next breath you mentioned that it would not be applicable to hot water, so I suppose that it dismisses it from use in water systems.

PROFESSOR UHLIG. —I have proposed this idea only as an example of basic research that holds promise of being useful practically, and which has not yet been evaluated in large-scale applications. Reduction of the corrosion rate by oxygenation of waters is useful only in cold-water systems, because as the temperature is raised, oxygen tends to attack the iron or steel locally, followed by deep pitting. Another way of expressing this is that at high temperatures passivity breaks down, and oxygen transforms from an inhibitor to an accelerator of corrosion. From fundamental considerations, we believe that this method applies to metals where passivity is possible.[2] Oxygenation, for example, would have no practical value where brass or copper piping is in use, because these metals are not passivated by excess oxygen as is iron.

* * *

UNIDENTIFIED. —In your laboratory experiments, was the aeration carried up to saturation?

PROFESSOR UHLIG. —We oxygenated the water. Oxygen from a commercial tank was bubbled through the water in which the iron specimens were immersed. Oxygen is required rather than air. Aeration will not produce similar passivity, except perhaps in the presence of metaphosphates. We are led to believe from present experimental data obtained at M. I. T., that the metaphosphates permit passivation of iron and steel at lower oxygen concentrations, and that this is one of the reasons why metaphosphates behave as corrosion inhibitors. The mechanism is related to the property of the metaphosphates as complexing agents. They form soluble calcium and magnesium salts and also soluble iron complexes. Ordinarily, when a piece of iron is exposed to water, FeO forms (hydrous FeO) with a layer of Fe_3O_4 on top, over which a layer of Fe_2O_3 normally forms. Practically all of these oxide layers are potentially soluble in presence of meta-

*Following Professor Uhlig's paper, several questions were asked by persons unidentified.

[2]Def. 1, p. 21, Corrosion Handbook, John Wiley and Sons, Inc., 1948.

phosphates, and hence they do not form and are no longer present to reduce the diffusion rate of oxygen to the steel surface. The dissolved oxygen in this situation has greater access to the clean surface of steel than normally, and is therefore able to chemisorb as a monolayer on the metal surface. This monolayer produces passivity in the same sense that chromates or similar materials are thought to make steel passive. The monolayer is essentially a dipole layer with negative charge outwards and with firm bonding to the metal surface so as to satisfy surface valences, thereby making the metal less reactive. If the barrier oxide films are present, it requires a much higher partial pressure of oxygen to achieve the same oxygen concentration at the surface of the steel, because oxygen at low partial pressures is rapidly consumed in reaction with nonpassive iron.

We are still in the process of evaluating this mechanism of inhibition by metaphosphates and similar complexing agents. Much of what I have said is not yet published, but I think that since the main purpose of this conference is to stimulate exchange of ideas, I intentionally injected these ideas and suggestions at this time.

* * *

JOHN F. WILKES.[3]—Professor Uhlig, would you care to elaborate on the protective action of nitrites in aqueous solutions?

PROFESSOR UHLIG. —Our feeling is that nitrites, like chromates, have chemical affinity for iron. There is no question of this, as tables of free energy bear out. However, the reaction of iron with nitrites is attended by a high-activation energy or, in other words, the reaction is slow. Because of the low rate with which chemical compounds form, the nitrite ion in the meantime can strongly absorb on the surface of iron as a monolayer or less of nitrite ions. As in the case of chemisorbed oxygen, substances of this kind set up a dipole layer with negative charge outwards and essentially satisfy surface valence forces, so that the metal surface is less reactive. We say, therefore, that the metal has been passivated by the nitrite, as determined by a more noble potential and a very low corrosion rate.

I should mention that there is another school of thought in this matter which ascribes to the nitrite ions the ability to oxidize ferrous oxides to supposedly more protective ferric oxides, the thought being that the higher oxide of iron is less permeable. These two points of view have not yet been resolved completely. Some of the basic work now going on at M. I. T. and elsewhere, is being done in an attempt to support one or the other point of view, and I have no doubt but that we shall soon achieve a better understanding of this subject.

[3]Technical Director, Dearborn Chemical Company, Chicago, Illinois.

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CHAS. H. SPAULDING.[4]—Professor Uhlig, one of the problems in the water systems is that there are so many different conditions in the systems, for example, where it is flowing fast and others where it is flowing slow; and then there are places where it is cold, places where it is warm and other places where it is hot, so that the equilibrium is upset, in one place or another. Conventionally deaeration has a promising possibility; I wonder if anyone knows of places that it has been applied, as a means of protecting the cold water systems in municipalities.

PROFESSOR UHLIG. —Not in municipalities, so far as I know. There are some industrial applications of deaeration for corrosion control both in this country and in Australia. In Australia, cold-water deaeration is used to mitigate corrosion of a steel main 30 inches in diameter and 350 miles long. By removing 90% of dissolved oxygen in the water, it was estimated that the life of the pipe would be increased by a factor of 4 times.

* * *

CHAS. H. SPAULDING. —I have in mind a couple of cases in which water was taken from a dead end, a line in which there was no flow, and in which the water was full of greenish iron. In other words, it had no oxygen, and this water was from a system which was supplied with saturated water with oxygen.

PROFESSOR UHLIG. —Stagnant water in the dead end of a pipe system may suffer increased corrosion through two factors. One factor relates to differential oxygen concentration which sets up a cell with positive pole or cathode in the well aerated section of the pipe, and the corroded electrode or anode in the poorly aerated section. Galvanic action of this kind usually occurs only when the conductivity of the water is relatively high.

Another factor may enter through the action of sulfate-reducing bacteria. The relatively low-oxygen concentration in the dead end of a pipe line stimulates the growth and reproduction of sulfate-reducing bacteria, which, in turn, accelerate the corrosion of iron. The corrosion products include hydrogen sulfide, iron sulfide and ordinary rust. Once established, these bacteria cause early perforation of the pipe line.

[4]Consultant, Urbana, Illinois.

It is also of interest that these bacteria may account for the active pitting of water pipes beneath tubercles. A tubercle is essentially a differential oxygen cell with a rust, calcium carbonate, or similar membrane separating a relatively aerated water outside, from poorly-aerated water inside. This alone can set up corrosion within the tubercle by differential oxygen concentration similar to corrosion at the end of the pipe line. In addition, sulfate-reducing bacteria may establish themselves inside the tubercle with corresponding high rates of corrosion.

* * *

JOHN F. WILKES. —Dr. Uhlig, in one of your earlier slides you showed the effect of pH on corrosion of steel. The data failed to take the curve down into the range of pH 13-14. I have encountered some very important problems due to corrosion, particularly in high-pressure boilers. I would like to hear more about the mechanism in that type of corrosion.

PROFESSOR UHLIG. --In the pH range 13-14, the corrosion of iron increases slightly, and the potential changes from a noble value typical of passive iron to an active value typical of nonpassive iron. The reason for this is that at high hydroxide iron activity, FeO no longer forms but instead a soluble sodium complex. This complex may be sodium hypoferrite or ferrite, which being soluble and tying up the ferrous ions in the complex, reduces ferrous ion activity at the metal surface. This accounts for the active potential and also the slight increase in corrosion rate.

At elevated temperatures, the rate with which alkalies react with iron, even at lower pH's, is very much higher. For this reason, the pH that can be tolerated in boilers is considerably less than 14. A curve showing the relation of corrosion rate to pH at 310° C. is given on page 525 of the Corrosion Handbook.

* * *

UNIDENTIFIED. —You mentioned carbon dioxide only to dismiss it early in your speech, and I would like a brief discourse on the part carbon dioxide has in the chemistry of corrosion.

PROFESSOR UHLIG. —The effect of carbon dioxide in the corrosion rate cannot be given in general terms. W. H. Walker and his associates were the first to demonstrate, in 1907, that carbon dioxide was not essential to the rusting of iron. This has been confirmed by later experience, and the early claims shown to be erroneous. The normal amount of carbon dioxide in natural soft waters, similarly, is not essential to the corrosion of iron. The corrosion rate-pH diagram shows that when the pH of a water becomes 4 or less, the corrosion of iron increases above that which applies to waters within the range of pH 4 to 9. When carbonic acid is present, rather than hydrochloric used in the experiments, upon which the pH curve is based, the critical pH at which hydrogen evolution occurs may be as high as 6. Therefore, only when carbon dioxide is present in amounts which depress the pH below 6 is the corrosion rate of iron affected.

In waters of positive Saturation Index, in accord with the Langelier treatment, the effect of carbon dioxide may be more appreciable than in the case of distilled or soft waters mentioned above. Dissolved carbon dioxide, in this case, may determine whether or not a water is undersaturated with respect to calcium carbonate and is then more aggressive, or whether it is supersaturated and therefore nonaggressive. A small amount of dissolved carbon dioxide can change a water from a so-called nonaggressive to an aggressive type, and may, therefore, determine whether a protective scale of calcium carbonate can form.

At high temperatures, another effect enters because the critical pH for corrosion attended by hydrogen evolution is appreciably higher than for waters at normal temperatures. In steam return lines, for example, dissolved carbon dioxide is very corrosive to iron and steel. As I started out to say, therefore, the situation is not simple, but depends very much on the conditions that apply.

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R. C. BARDWELL.[5] —I should like to ask how this threshold treatment, by the addition of a few ppm. of hexametaphosphate, affects the Saturation Index.

PROFESSOR UHLIG. —The Saturation Index no longer applies when hexametaphosphates are present to the extent that calcium and magnesium salts are changed to soluble complexes. The Saturation Index development depends upon the formation of insoluble protective calcium carbonate films on the metal. Therefore, when these protective films can no longer form, the Langelier treatment does not apply.

[5]Engineer of Tests, Chicago and Eastern Illinois Railway, Danville, Illinois.

THE COMPOSITION OF WATER SUBSTANCE[1]

BY W. H. RODEBUSH*

The ideas presented in this paper represent the conclusion of fifteen years of cooperative research between the Illinois State Water Survey and the Division of Physical Chemistry of the University of Illinois. The details of this research have been or will be published elsewhere with due acknowledgements to individuals and to agencies for financial assistance.[2]

The research had its inception in discussions between Dr. A. M. Buswell and the author regarding the nature and composition of water with particular reference to the living organism. Dr. Buswell found certain generally accepted concepts to be so vague as to actually constitute a barrier to further progress in the field. As always in situations of this kind, fundamental research is the only solution.

For example, water was commonly believed to be an "associated" liquid but no one seemed to be able to define what was meant by associated. One authority, for example, stated that liquid water was "dihydrol" and ice "trihydrol." Another ill-defined concept was that of "bound" water. This term was perhaps never in good repute and we now know that it is misleading—free water would be more exact. But the phenomena in living organisms that are implied by the term are of great importance. For example, the economic importance of the frost resistance of wheat and corn are obvious. Professor Gortner at the University of Minnesota had shown that, of two samples of wheat showing the same percentage of water by the ordinary methods of analysis and the same osmotic coefficient, one would winterkill and the other would not. It is clear that the principle involved here is something other than the one by which automobile radiators are protected by anti-freeze.

The fact that the association of water is due to hydrogen bonding had been announced by Latimer and Rodebush[3] in 1920 but no direct experimental evidence for this had been produced until the discovering in 1934 by Wulf and his collaborators[4] that the second harmonic in the infrared absorption of the hydroxyl group in certain molecules became very diffuse when these groups formed hydrogen bonds. It was obvious that infrared spectroscopy was a very promising method of approach to the quantitative study of hydrogen bonding. An infrared apparatus was built in this laboratory and an important discovery was made.[5] In addition to confirming the effect observed by Wulf et al., it was found that the fundamental absorption of hydroxyl was shifted to longer wave lengths and the extinction coefficient greatly augmented when a hydrogen bond is formed. This discovery was made independently by Errera.[6] This effect is proportionate to the acidity of the hydrogen as was announced by Venkateswaran.[7]

The infrared spectrometer immediately became a tool of the greatest utility, particularly to the organic chemist. Some fifteen papers were published from this laboratory dealing with structure as a result of infrared studies. One of the important results established was that the characteristic cross linkage in proteins between peptide chains is the NH→O linkage.[8]

The attempt to determine the degree of association of water by the use of infrared was, however, doomed to failure. Water is so opaque to infrared even in the thinnest layers that no quantitative measurements could be made. The higher harmonics of the hydrogen bond absorption certainly extend into the visible region, giving an absorption in the red or orange and producing the bluish-green color of water in thick layers. The determination of the degree of association of liquid water must be made by more indirect methods, and the best approach can be made by a comparison of the structures of ice and water.

Ice-Water Structure. The layman has no difficulty in distinguishing ice from water, and it comes as a surprise to realize that the only properties which are essentially different for these two phases

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[1]The title is suggested by the excellent monograph entitled, "The Properties of Ordinary Water Substance," by N. E. Dorsey, Reinhold Publishing Co., New York, 1940.

[2]Acknowledgement for financial support is due to the Rockefeller Foundation, to the United States Public Health Service, contract no. 666 and to the Office of Naval Research, contract no. N6 ORI-71.

[3]W. M. Latimer and W. H. Rodebush, J. Am. Chem. Soc., 42, 1419 (1920).

[4]G. E. Hilbert, O. R. Wulf, J. B. Hendricks and U. Liddell, J. Am. Chem. Soc., 58, 548 (1936).

[5]A. M. Buswell, V. Dietz and W. H. Rodebush, J. Chem. Phys., 5, 84 (1937).

[6]J. Errera and P. Mollet, Comptes Rendus Acad. Sci., 204, 259 (1937).

[7]C. S. Venkateswaran, Proc. Indian Academy, 7, 13 (1938).

[8]A. M. Buswell, J. R. Downing and W. H. Rodebush, loc. cit.

are density and viscosity and other properties depending upon these properties. The infrared absorption, the dielectric constant, and the structure itself changes continuously through the melting point so that the melting point might almost appear as a second order transition. Even the energy difference between ice and water is so small that it seems to be an accident as to which is the stable phase at lower temperatures.

Water is one of the few substances for which the liquid phase is more dense than the solid, and it is probably the only substance that exhibits a temperature of maximum density a few degrees above the melting point. It is interesting to consider the reason for this behavior. Ice is a hexagonal crystal. The water molecules have a coordination number of four, each water molecule being hydrogen bonded to four other water molecules in tetrahedral arrangement. The distance between oxygen atoms is 2.76 Å. The structure of ice is very open, hence the low density. There are two hydrogen bonds per molecule, and if we lump all the attraction forces between molecules into one package and call it the hydrogen bond, then the energy per bond is about 6000 calories per mole. When ice melts to water, the distance between oxygens increases to about 2.90 Å.[9] This would cause an increase in volume and corresponding decrease in density of about 15%. It would also decrease the energy of the hydrogen bonds by 25% or more, and we should expect a heat of fusion of 3000-4000 calories. The heat of fusion is only 1440 calories per mole, and this is explained by the fact that additional hydrogen bonds are formed. The very open structure that is formed permits an increase in coordination number to a maximum of five or six, which, of course, accounts for the increase in density. The additional water molecules are hydrogen bonded. One pair of electrons bonds two hydrogens through a resonance mechanism such as Pauling has postulated for the metals. This resonance mechanism is greatly favored by an increase in ionic character of the O-H bond in the water molecules. Thus water has more hydrogen bonds than ice, even at the boiling point.

The heat capacity of liquid water is more than twice that of ice. The behavior of water in this respect is certainly different from that of any other liquid, and it has commonly been stated that the high value is due to the breaking of hydrogen bonds. A simple calculation shows that this explanation is untenable. The fact that the heat capacity is constant over the whole range from the melting point to the boiling point, within a few per cent, can be given a simple explanation. For an ionic compound we have equipartition so that the hydrogens and the oxygen give the Dulong Petit value just as sodium and chlorine in sodium chloride.

The open structure of ice resembles a bridge arch under heavy stress. The internal pressure of liquid water due to the attraction forces between molecules would amount to a couple of thousand atmospheres if the liquid had the density of ice. Under the thermal agitation of these molecules, the ice structure collapses at 0° C. It is well known that the melting point is lowered by pressure, and we may assume that if the internal pressure were removed from the liquid, ice would not melt unless heated to 15 degrees or more centigrade above the melting point. We shall want to keep this fact in mind a little later in this paper.

Although the heat of vaporization of water is much greater than benzene, the heat and entropy of fusion are only a little more than half that of benzene. This is because water has a structure, more structure, in fact, than any other true liquid. If water could be supercooled, it would form a glass, but the hydrogen atoms are so mobile that the supercooling cannot be carried beyond a few degrees.

The structures postulated by Pauling to give a residual value of entropy to ice at low temperatures have nothing to do with a glass-like structure. They are variations in the definite crystalline arrangement of the hydrogens, which at low temperatures would be frozen into one configuration. At higher temperatures enough hydrogen would be dissociated to insure a "flexible" structure for ice.

The Physical Properties of Water. The physical properties of water such as viscosity and dielectric constant are most readily understood when the properties of the liquid are contrasted with those of ice. These properties involve what are known as cooperative phenomena, that is, the displacement or orientation of a series of hydrogen atoms which are attached by a covalent bond to the oxygen of one molecule and hydrogen bonded to the oxygen of another molecule. If this displacement or transfer of bonds can take place one at a time—by what may be described as "zipper" action, the forces required are small but the action is slow. If a rapid displacement and transfer of a series of hydrogen bonds is to occur, then very great stresses must be applied so that a simultaneous breaking of bonds is possible. The significance of the foregoing consideration becomes apparent on a detailed consideration of the mechanism of viscosity and dielectric polarization.

Viscosity. The viscosity of liquid water is not greatly different in magnitude or temperature coefficient from that of other liquids. The same mechanism is involved; the molecules must slip past one another—"jump over one another" is a more graphic expression. This involves a heat of

[9] T. J. Morgan and B. E. Warren, J. Chem. Phys., 6, 666 (1938).

activation which may vary all the way from zero to a few thousand calories. Because of the structure of water, the process is more uniform and the heat of activation varies within narrower limits. The temperature coefficient of the viscosity of water is about 2% per degree negative, which indicates an activation energy of 3600 calories per mole per hydrogen bond. This is less than the energy of dissociation of a hydrogen bond because of the largely ionic character of the bond. A hydrogen may be attracted to two oxygens at once or even three. It should be recalled that with a coordination number of 6 in the liquid, each hydrogen must be surrounded by three oxygens. The transfer of the proton from molecule to molecule is facilitated and the energy hump to be traversed is greatly reduced.

In the case of ice, no such transition is possible. The hydrogen bond must be completely dissociated before any motion can occur since the hydrogen has nowhere to go—hence the greatly higher viscosity of ice. While enormous pressures are required to crush the ice structure and lower the melting point, ice will flow quite readily under shearing stresses. This is because the stress is applied to a relatively few hydrogen bonds along the glide surface and the mass breakage of these bonds becomes possible since only a few bonds are strained at any instant.

The displacement of the hydrogen bond from one oxygen to another may be represented as the surmounting of a potential energy barrier approximately equivalent to 3600 calories activation energy. The fraction of hydrogen bonds having sufficient energy to transverse this barrier is given by the Boltzmann factor $e^{-\frac{Q}{RT}} \sim 10^{-3}$ at 25° C. Since the frequency factor for this transition is about 10^{13}, we have therefore a probability that the hydrogen bond will transfer 10^{+10} times per second. The movement of a water molecule involves the transfer of two hydrogen bonds, but the process is a very rapid one. It is, however, as rapid in one direction as the other. This is in the absence of a flow. When a pressure gradient is applied, a bias is introduced into the two energy levels on either side of the barrier, tending to raise the one and depress the other. The result is that the rate of transfer is greater in the one direction than the other and there is a flow, which will be shown later in the analogous case of the electrical field to be proportional to the pressure gradient.

There is a finite limit to the number of transfers occurring per unit time, and when the pressure is increased the height of the barrier is increased because of the closer approach of the atoms so that the flow no longer increases linearly with the pressure gradient. Further increase in the stress results in turbulent flow. The situation becomes analogous at least to the flow of ice under shearing stress, where bonds are actually ruptured by the applied force. As the forces are increased further, there is reduction in apparent viscosity because of the rupture of bonds so that the flow is no longer newtonian once the critical velocity of flow is exceeded.

This sharp distinction between laminar flow and turbulent flow may be described in terms of the transfer of hydrogen bonds. As long as the dissociation is simply a transfer corresponding to the normal rate as given by the kinetic theory and the process may be described as adiabatic, there is very little heat evolved. When the process becomes violent, we have rupture of bonds rather than transfer, the water behaves as a rigid substance, a great deal of heat is produced, and a characteristic turbulence results. It would be very informative to measure the rates of flow of ice and water under conditions of carefully controlled stress.

Dielectric Polarization. The Debye theory of rotating molecules cannot be correct for a polar liquid, as has been shown by Onsager, and it does not give the correct temperature coefficient. We are dealing with a cooperative process in which a group of molecules is oriented. Since ice has, near the melting point, a dielectric constant similar to that of water, we can assume that the polarization of ice is similar to that of water. There is no difficulty in the progressive orientation of water molecules. The process is similar to the laminar flow described above. In the case of ice, the situation is very different. A few of the hydrogen bonds are, of course, dissociated, the fraction being given by the Boltzmann factor $e^{\frac{Q}{RT}} \sim 10^{-5}$ at 0° C. The polarization of ice must occur by a progressive orientation starting from these dissociated bonds as centers. This seriatim process is necessarily slow. The lower the temperature, the slower the process, so that at -20° C. the maximum, in phase current is obtained with a frequency of only 1000 cycles per second. At still lower temperatures, the polarization process does not occur at all, so that the dielectric constant falls to a low value. It will be seen that simultaneous studies of dielectric constant and viscosity would prove informative.

Water as a Solvent. When we consider the properties of water as a solvent, the parallel between ice and water no longer holds. This is because the rigid lattice of ice, as we shall see later, allows no cavity large enough to hold even the smallest molecules. Water is conspicuously and par excellence a solvent for electrolytes. For more than sixty years, since Arrhenius announced his theory of electrolyte dissociation, physical chemists

have studied electrolyte solutions. There have been long and bitter disputes and polemics. G. N. Lewis devoted some of the best years of his life to untangling the confusion between activity coefficient and degree of dissociation. The laboratory of physical chemistry at the University of Illinois was dedicated, when built, to the study of ionic migration. The final grist of this mill of the Gods is a fine powder indeed. The Debye-Huckel theory holds for simple electrolytes at concentrations below one hundredth normal!

But all of this study has told us nothing about water. The reason for this is that when one dissolves an electrolyte in water, one destroys the structure of water because of the ionic attraction forces. The truly bound water is to be found in a salt solution.

Solutions of Nonpolar Substances. The distinction between polar and nonpolar molecules can be made in terms of the field surrounding the molecule. Both types of molecules are surrounded by electrical fields, but the field around a polar molecule falls off as the inverse second or third power, while the field around a nonpolar molecule falls off very rapidly as the inverse sixth or seventh power. In other words, polar molecules are capable of action at a distance and nonpolar molecules exhibit only contact forces. Because contact is limited, the attraction between nonpolar molecules is less than the attraction between polar molecules. If we place a nonpolar molecule in a polar environment, there will be a minimum of attraction between it and the environment. One would expect the solubility to be slight, and this turns out to be the case so that there is nothing remarkable here. The interest in these solutes would have been negligible were it not for the existence of hydrates of nonpolar molecules. These hydrates were called to the attention of the chemist in a dramatic fashion by the fact that natural gas lines were observed to become clogged when water was present at temperatures as high as 20° C. Methane is a very nonpolar molecule, but even more striking is the fact that argon, krypton and xenon form hydrates. The noble gases form no compounds, and the explanations that have been offered for the existence of these hydrates are fantastic indeed. Of course, the real explanation for the existence of these hydrates is to be found not in attractive forces between water and the nonpolar molecule, but in the lack of attraction. As an approach to the problem of the nonpolar molecule hydrates, we began in 1946 the study of the solubilities of particular hydrocarbons in water. If one measures both the solubility and its temperature coefficient, one obtains the data from which one may calculate the heat of solution, free energy of solution, and the entropy of solution of the substance. The results obtained were sufficiently striking, and it will suffice here to describe the behavior of two hydrocarbons.

I. When methane dissolves in water, the heat liberated is ten times what it is in hexane although the solubility in water is only one tenth what it is in the hydrocarbon.

II. The heat of mixing of benzene and water is zero just as is the heat of mixing of benzene and toluene, but benzene and water are almost immiscible. Thermal neutrality is taken as a criterion for an ideal solution, and benzene and water show the extreme possible deviation from ideality.

These facts become more surprising on close examination. A methane molecule occupies a volume in water solution of about 60 $(\text{Å})^3$ rather greater than the volume of two water molecules. According to simple theory, the formation of a cavity of this size in water should require an energy somewhat greater than the heat of vaporization of water, say 15,000 calories per mole. No one can suppose that the attractive forces between methane and water can furnish any appreciable part of this energy.

Likewise, in the case of benzene, the heat liberated when one cubic centimeter of liquid is formed is about 70 calories, while the heat liberated in the formation of one cubic centimeter of liquid water is 500 calories. It is difficult to see how benzene and water could be mixed without a considerable heat effect. There are two possible explanations for these effects. One must assume either strong attractive forces between the water and the hydrocarbon or profound changes in the water structure itself which are induced by or allowed by the presence of the hydrocarbon.

Let us consider the latter alternative first. It is not certain who first propounded this theory, but it has been termed the "iceberg" theory since it assumes that each nonpolar molecule is surrounded by an envelope of water molecules which are essentially "frozen" or crystallized.[10] Such a structure involving ten or twenty water molecules would allow for the decrease in energy and entropy that must be assumed to explain the solubility of hydrocarbons. The formation of "ice" at an interface with a hydrocarbon molecule may be explained as due to the decrease in internal pressure because of the lack of attractive forces. Under these conditions the freezing point might be raised to a considerable amount, although it cannot be supposed that the structure of the shell of water molecules resembles that of ice very closely.

If the iceberg hypothesis is to be accepted, one should be able to show that the partial molal volume of the hydrocarbon should be larger in water than in other solvents. Reports in the literature indicate that this may be so, but more accurate data

[10] H. S. Frank and M. W. Evans, J. Chem. Phys., 13, 507 (1945).

are required. We have developed an apparatus for measuring densities to one part in one hundred million, which should give us confirmation on the point.

There is, of course, an alternative to the iceberg theory which must be given serious consideration. Preliminary results obtained on the density of benzene indicate that the partial molal volume of benzene in water is about 83 cm.^{3}, less than it is in liquid benzene. This indicates that benzene is compressed in water solution and that a very intimate contact exists between water and benzene. There seems to be no chance for an iceberg theory here, and the fact that benzene forms no hydrates is in line with this thinking. Actually, the electronic constitution of benzene suggests that benzene in water solution is actually hydrated through hydrogen bonding, although confirmation of this has not been obtained. At any rate, we must assume a far greater attraction between benzene molecules and water molecules than between two benzene molecules. That this is so is an experimental fact. Since the van der Waals energy varies as the inverse sixth power (V^{-2}), we can account for a strong attraction between a benzene molecule and the surrounding water molecules. What we cannot estimate with certainty is the energy required to form the cavity occupied by the benzene molecule. If one calculates the surface energy of the equivalent spherical cavity, one obtains 9000 calories, which seems a very small amount, but the attempt to treat so small a cavity as having a continuous surface energy is a very dubious procedure. It is still more difficult to account for the decrease in entropy that limits the solubility to such a small value but some of this can be attributed to the compression of the benzene molecule and the remainder is perhaps to be found in the hydrogen bonding of water molecules to the benzene.

The existence of solid crystalline hydrates of most of the smaller nonpolar molecules lends support to the iceberg theory. One can imagine that if the concentration of hydrated molecules is increased, then at some concentrations there will be a tendency for a crystalline hydrate to separate out, provided the temperature is not too far above 0° C. The individual molecule in solution will be surrounded by a cluster of perhaps twenty water molecules but when the crystalline hydrate is formed, the sharing of water between two nonpolar molecules will reduce the hydrate ratio to a much smaller number. The fact that benzene does not form a hydrate indicates that the alternative explanation must be sought for the behavior of benzene solutions.

Interfacial Hydrates. Let us first observe that while there is a strong attraction between electrolytes and water, there is little tendency toward hydrate formation. Many salts crystallize in an anhydrous state, and such hydrates as are formed appear to occur only when the lattice is such as to allow accidental inter-lattice spaces to be filled with water. On the other hand, practically all of the nonpolar molecules form hydrates with, in many cases, a large number of molecules of water. We believe that these hydrates are due not to an attraction between the nonpolar molecule and the water, but to lack of attraction, and the hydrate is due to a phenomenon involving crystallization of the water at the interface between the water and the nonpolar molecule.

It was mentioned earlier in this paper that the enormous internal pressure existing in liquid water must actually lower the melting point of ice by some 15° C. Since this pressure does not exist at the surface of the liquid, one might expect crystallization in the surface at temperatures of 15° to 20° C. This may occur, but if it does, it escapes observation because it is so thin. In a solution of nonpolar molecules in water there is an interface surrounding each molecule, across which only weak forces act at high concentrations. In other words, a continuous phase of water surrounds the dispersed phase of nonpolar molecules under very low internal pressure. Under such conditions the water freezes. This is our picture of the nonpolar hydrates.

Structure of Nonpolar Hydrates. In order to understand these interfacial hydrates, it is necessary to consider the exact molecular structures. A very large number of nonpolar hydrates are known, of which a few examples are listed herewith. It is seen that they tend to fall into groups although it must be admitted that the data are not too exact, since most of the determinations were made more than fifty years ago. Because these hydrates are not too well defined as crystals, they tend to include water and extraneous gases so that exact determinations of the number of water molecules is a very difficult matter. The hydrate numbers appear at first so odd as to be improbable. It turns out, however, that these are precisely right in some cases at least.

Ice forms a hexagonal crystal of which the unit structure is eight water molecules arranged tetrahedrally in a series of fused hexagons resembling the benzene ring. These molecules form a cage which is too small, however, to contain a gas molecule. The structure of the hydrates must be assumed to resemble ice in that the tetrahedral arrangement must be preserved. The structure must be open enough to enclose gas molecules, and X-ray data show the crystal has a cubic symmetry. We sought, therefore, to construct a cage of tetrahedrally connected water molecules. The first requirement is that this cage must be large enough to contain a gas molecule, and the second require-

Table I

Physical Data on the Two Cubic Hydrate Lattices

	Diamond Type	Body-centered Type
Cell constant (approx.)	17 Å	12 Å
Number of H_2O molecules in cell	136	46
Number of dodecahedra (small holes)	16	2
Number of 14-hedra (medium holes)	none	6
Number of 16-hedra (large holes)	8	none
Hypothetical hydrative numbers		
(a) Filling all holes	136/24 = 5-2/3	46/8 = 5-3/4
(b) Filling only larger holes	136/8 = 17	46/6 = 7-2/3

* * * * * * *

Examples of Hydrates of Small, Medium and Large Inert Molecules

Small 5-3/4 H_2O	Medium 7-2/3 H_2O	Large 17 H_2O
$C_2H_2 \cdot 5.7\ H_2O$	$C_2H_6 \cdot 7\ H_2O$	$CCl_4 \cdot 16\ H_2O$
$PH_3 \cdot 5.9\ H_2O$	$C_2H_4 \cdot 7.4\ H_2O$	$CHCl_3 \cdot 17\ H_2O$
$H_2S \cdot 5.7\ H_2O$	$C_2H_5F \cdot 8.27\ H_2O$	$CH_2Cl_2 \cdot 16\ H_2O$
$H_2Se \cdot 5.87\ H_2O$	$CH_3Cl \cdot 7.2\ H_2O$	$C_3H_8 \cdot 17\ H_2O$
$CH_4 \cdot 6.3\ H_2O \longrightarrow$		
Body-centered cubic - dodecahedral lattice		Diamond-dodecahedral lattice

ment is that structures be completely hydrogen bonded. This means that the hydrogens must be turned outward with the oxygens on the inside. Dr. Buswell had suggested that the pentagonal dodecahedron, Fig. 57, was a promising structure. Dr. W. F. Claussen of this laboratory attacked the problem of building these structures into a repeating lattice and with great ingenuity succeeded in this project. Some idea of the persistence and ingenuity required is given by the fact that 136 molecules are required to form the unit cell. Dr. Claussen started out working with models trying to see how the structure could be combined with a repeating lattice. The angles of the pentagon are 108°, only a slight distortion from the well known tetrahedral angle. Dr. Claussen was greatly aided in his task by the work of von Stackelberg[11] in Germany, who had already determined the lattice spacing for some of these hydrates but who had been unable to imagine what kind of a structure could be involved.

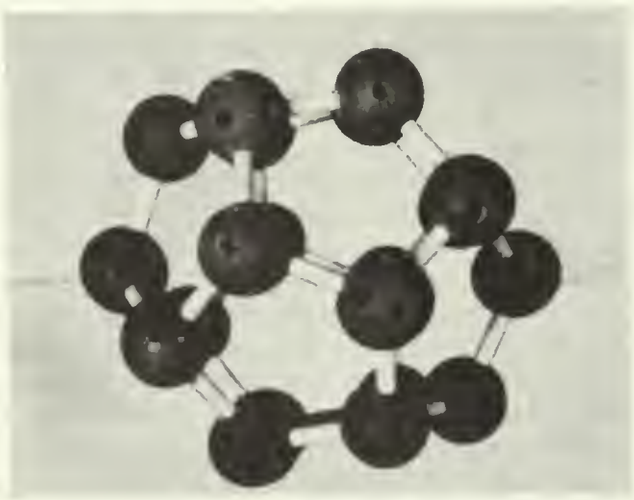


FIG. 57. —PENTAGONAL DODECAHEDRON.

The way in which these two methods of packing dodecahedra were discovered was in each case to locate dodecahedra at lattice points in two of the

[11] M. S. Stackelberg and H. R. Muller, J. Chem. Phys., 19, 1319 (1951).

FIG. 58.—BODY-CENTERED CUBIC PENTAGONAL DODECAHEDRAL LATTICE SHOWING BASE AND CENTER OF UNIT CELL.

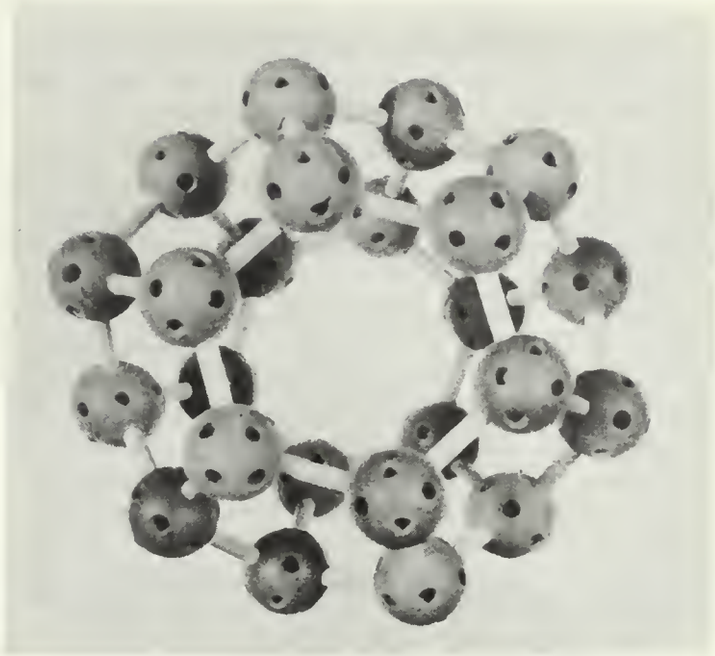


FIG. 59.—TETRAKAIDECAHEDRON.

familiar cubic lattices, the body-centered cubic lattice and the diamond cubic lattice. In the former, the body-centered, a pentagonal dodecahedron is located at each corner and in the center of the cubic unit cell. One might term this lattice, the pentagonal dodecahedral body-centered cubic lattice, Fig. 58. Besides the dodecahedral water molecules, a few more were added to make up the unit cell; these served to join together the "corner" dodecahedra and to complete planar hexagons of water molecules at these junctures. Out of this structure came a new void, a 14-hedron (or tetrakaidecahedron), Fig. 59, which has two opposite hexagonal faces and twelve pentagonal faces. This new void is slightly larger in volume than the dodecahedral void, and thus it could accommodate a slightly larger inert gas molecule than could the dodecahedron.

The second hydrate structure involved the pentagonal dodecahedron and the diamond cubic lattice, two opposite water molecules of the dodecahedron being superimposed on two lattice points in the diamond cubic lattice.[12] This superposition is possible by deforming the angles around these two opposite water molecules to make them exactly tetrahedral. Then the symmetry at these points is identical to that found in diamond. The resulting hydrate model produced still another void, a 16-hedron (hexakaidecahedron), Fig. 60, consisting of four hexagonal sides and twelve pentagonal sides. This void is larger than either the pentagonal dodecahedron or the 14-hedron. Thus, we have three sizes of voids and two hydrate lattices. One can imagine some inert molecules being too large to get into the smaller holes and even too small to stay in the large holes. One can imagine filling the large holes with large molecules and the small holes with small molecules to form a mixed hydrate of the diamond type; such a structure has been observed in $CHCl_3 \cdot 2H_2S \cdot 17H_2O$. One can imagine that perhaps only the large holes will be filled, the small ones remaining empty. This has been observed in $CHCl_3$ hydrate. One can imagine, for the body-centered cubic dodecahedral lattice, the medium holes (14-hedra) being filled by medium molecules and the small holes (dodecahedra) remaining empty. This has been observed for ethane hydrate. One might fill both the small and the medium holes of the body-centered lattice with small molecules; this has been observed for H_2S hydrate. One might fill the medium holes with medium-sized molecules and the small holes with small molecules; this has never been reported.

One might fill all of the holes of the diamond lattice with small molecules. This has not been found and probably cannot exist because the larger holes, the 16-hedra, of this lattice are just too large for the small molecules. Another way of stating it is that the lattice containing the small and the medium holes is favored for small molecules alone over the lattice containing the small and the large holes because of energy considerations, and therefore, if any hydrate of the latter is formed, it will tend to decompose and be reformed into the former type.

The significance of the interfacial hydrates in the living organism cannot be overemphasized. Just as life itself as we know exists or is dependent upon

[12] A similar or perhaps identical structure has been suggested by L. Pauling in a letter dated June, 1951.

a land-water boundary, so the processes of life occur mainly at the interface between water and the protein molecule. The protein molecule contains large nonpolar groups and the tendency for water to crystallize in the presence of these groups is very great. The hydrate so formed has a lower density than ice so that large volume changes can be induced here.

Certain varieties of corn frost when the temperature drops to 40° F. This becomes understandable in terms of the formation of a hydrate at temperatures well above the freezing point. On the other hand, the wheat plant growing through the fall season with progressional lowering temperatures can form the hydrate slowly and without damage to cells, thus forming a most effective antifreeze. The significance of these hydrates to the frozen food industry remains to be exploited.

The existence of the hydrates is, of course, a convincing argument for the "iceberg" theory of the solution of nonpolar molecules, but the hydration number in solution will bear no relation to that in the crystal. As has been stated above, it will in general be much larger; thus a single dodecahedron contains twenty water molecules. The direct determination of hydration in solutions is as yet an unsolved problem, although the physical chemistry laboratory in Noyes Laboratory, built in 1916, was dedicated by E. W. Washburn to the solution of this problem. More of this later. The most direct attack upon the problem of hydration appears to us to determine the changes in density of water upon solution of nonpolar molecules

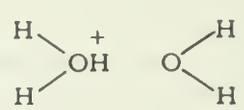
The Hydration of Ions. The most reliable data from which to calculate ionic hydration are the ionic mobilities. Assuming Stokes' law to hold, the mobility at unit field strength varies directly as the charge and inversely as the diameter and the viscosity of the solvent. The effective diameter includes, of course, the attached water molecules, but if these do not form a smooth spherical shell, they are likely to oppose a variable resistance to movement through the liquid and to be detached and reunited to the ion. The familiar and at first surprising fact is that the ions that are known to be of smallest radius have the lowest mobilities. This is, of course, because they are hydrated to the greatest extent. A comparison of the mobilities of the various ions leads to the following conclusions.

1. Singly charged cations larger than potassium are unhydrated.
2. Most anions are unhydrated.

It is simpler to consider the anions first. An anion can only hydrate through forming a hydrogen bond. Oxygen and fluorine are the only atoms capable of acting as electron donors; hence fluorine and hydroxyl are the only singly charged anions likely to be hydrated. In any event, not more than one or two water molecules will be attached.

The smallest cation is the hydrogen ion, and it is undoubtedly the most hydrated and would have the lowest mobility were it not for the proton transfer effect. Since hydrogen and hydroxyl ions appear anomalous from the standpoint of their mobility, let us consider them first. As stated above, the mobility of an ion should vary inversely as the viscosity, and the temperature coefficient of the mobility of most of the ions is exactly what would be calculated from the temperature coefficient of viscosity. In the case of the hydrogen and hydroxyl ions, the temperature coefficients are respectively 0.016 and 0.018, corresponding to heats of activation of 2800 and 3200 respectively. It is interesting to see what these heats of activation may signify.

Freshman students of chemistry have been mystified for many years by being taught that the formula for the hydrogen ion is H_3O^+. It would be more correct to say that the formula is $H_5O_2^+$, since the exact structure must be



The hydrogen ion is nearer one oxygen than the other, but, as we shall see, it oscillates between the two oxygens many times per second. Likewise, the formula for the hydroxyl ion has the structure

$$HOH \quad \overline{O}H$$

and here the proton oscillates between the two oxygens again with the high frequency.

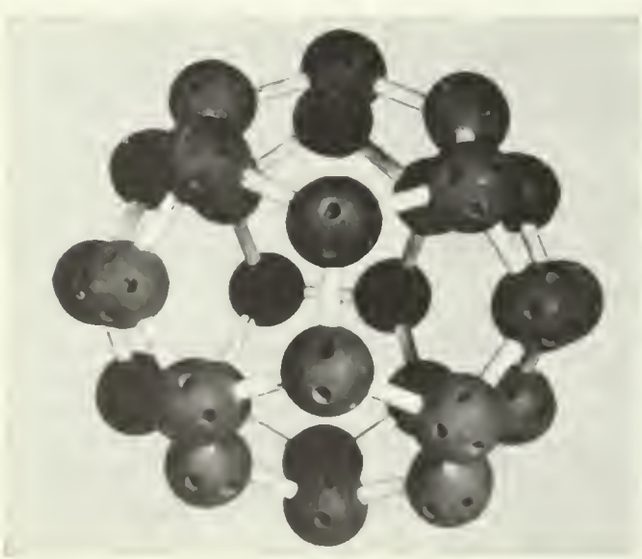


FIG. 60. —HEXAKAIDECAHEDRON.

Low Conductance of Liquid Water. The electrician who knows the danger of moisture when handling live wires will be skeptical about the low conductance of water, but the fact is that pure water is a good insulator. The common impurities such as carbon dioxide in the atmosphere dissolve to give hydrogen ions and hence the high conductivity.

It is necessary, therefore, to explain the low conductivity of pure water as an ionic substance, and here the analogy to the salt crystals is helpful. The pure salt crystal has practically no conductivity. If the lattice were perfect with an exactly equivalent number of positive and negative ions, there would be no conductance through the mass of the crystal. (Surface conductance is, of course, another matter.) The energy of activation required to put a series of ions in motion would increase with the dimension of the lattice in the crystallite. Lattice imperfections are likely to exist at any temperature above zero Kelvin, and one of the commonest ones is for an ion to be lacking, leaving a hole unfilled. This may be due to a deficiency of ions of one sign, but it may equally well be due to displacement, so that an ion of opposite sign will be missing somewhere else in the lattice. This corresponds to the ionization of pure water. Under the driving force of an electric field an ion of the charge sign of the missing ion will slip into the hole, and the hole will migrate in the opposite direction to the ion. This gives a perfect illustration of what has been termed zipper action in the discussions of cooperative phenomena.

The behavior of water is precisely analogous to the salt crystal so far as electrical conductivity is concerned. So long as hydrogens and oxygens are present in equivalent amounts, the conductivity is negligible even though the hydrogen-oxygen linkage in water is largely ionic in character. If a single hydrogen ion breaks loose from an oxygen atom and starts to move under the influence of the field, it can get nowhere because it will be drawn back to the negative ion. In order to get conductivity in this manner, it would be necessary to dissociate a whole series of hydrogen atoms through the solution, and this means an enormous energy of activation. This was the original mechanism suggested by Grotthus for electrolytic conduction. Such conductance as pure water has, therefore, is due to the spontaneous ionization into hydrogen and hydroxyl ions which are completely separated from each other. The mechanism of conduction is therefore due to a local excess or deficiency of an ion of one sign or the other, precisely as is the case with the salt crystals. It is possible to calculate the mobility of the hydrogen and hydroxyl ions with considerable precision. The initial step is the proton jump between two water molecules in the $H_5O_2^+$ complex. So long as there is no field present, the oscillation of the proton between the water molecules is occurring with a frequency

$$\nu e^{-\frac{Q}{RT}}$$

where Q is the activation energy for the jump 2800 cals./mole, and ν is the frequency of the proton oscillation in H_3^+O, known from infrared absorption to be of the order of 10^{14}. So long as no field is present, the structures are symmetrical but when a field is applied, a bias is produced in the energy levels of the proton in the two water molecules, which is

$$\frac{EfZ \cos \Theta}{4.18} = 23060\ EZ \cos \Theta$$

where E is the field strength, f the Faraday, Z the distance of the proton jump, and Θ the angle with the electric field. The factor 4.18 converts volt equivalents to calories. This situation is mathematically the exact analogue of the cases described earlier in that a bias in the energy levels is produced by a pressure gradient or a polarization field. It was stated thus that for moderate values of the applied stresses the effects are proportional to the applied stress, and we may proceed to show why this is so in the case of the hydrogen ion conductance.

In the presence of the applied field the expression for the rate of the protons' jump with the field is modified to

$$\nu e^{-\frac{Q - 1/2 \times 23060\ EZ \cos \Theta}{RT}} \qquad \text{(I)}$$

while the reverse process is

$$\nu e^{-\frac{Q + 1/2 \times 23060\ EZ \cos \Theta}{RT}} \qquad \text{(II)}$$

The net transfer of protons is the difference between the two expressions. For small values of E,

$$e^{\pm \frac{23060\ EZ \cos \Theta}{2\ RT}}$$

becomes

$$1 \pm \frac{23060\ EZ \cos \Theta}{2\ RT}.$$

Hence (I) - (II) becomes

$$\nu e^{-\frac{Q}{RT}} \left[\left(1 + \frac{23060\ EZ \cos \Theta}{2\ RT}\right) - \left(1 - \frac{23060\ EZ \cos \Theta}{2\ RT}\right)\right]$$

and for random orientations the average value of $\cos \Theta$ is 1/3. The rate of proton transfer by jump-

ing from one molecule to another is therefore

$$\nu e^{-\frac{Q}{RT} \times \frac{23060\ EZ}{3\ RT}}$$

The apparent mobility of the proton is increased, however, by the possibility of proton exchange in the molecule. When a proton jumps to a molecule to form the complex $\overset{+}{HO}\begin{matrix} H \\ H \end{matrix}$, the proton which leaves this molecule is the one which is on the farthest side of the molecule in the direction of the field. The total gain in distance per jump therefore is more nearly three Angstroms. Hence, substituting $Z = 3 \times 10^{-8}$, $E = 10^{-8}$ and remembering that the mobility is the rate of transfer at unit field strength we have for the mobility

$$10^{+14}\ e^{-\frac{2800}{600} \times 10^{-8} \times 10^{-8} \times \frac{23060}{600}} = 3.8 \times 10^{-3}$$

For the hydroxyl ion the same calculation gives 1.6×10^{-3}, using $Q = 3200$ calories. The agreement is better than one has any right to expect but there is no doubt about the order of magnitude. Each hydrated ion complex has a mobility of its own independently of the proton jump mechanism. In the case of the proton this is small, but the hydroxyl is not greatly hydrated and must therefore contribute considerably to the conductance.

The reason that anions do not form large envelopes of water molecules is that the hydrogen bond does not "propagate" through several water molecules. The formation of a hydrogen bond does not polarize the water molecule. On the other hand, a cation attracts the oxygen of the water molecule and makes the hydrogens more acidic. They bond to oxygens of other water molecules so that the polarization extends through at least two layers and may involve a total of ten or more water molecules. This polarization is greater the greater the charge, and the smaller the radius of the cation. In the case of the small univalent cations this theory receives confirmation from the activity coefficients. In concentrated solutions the activity rises to values greater than unity because the concentration has been increased to a far greater value than the nominal value on account of the water of hydration.

The larger univalent cations, such as potassium, for example, coordinate two water molecules, but loosely, and the volume of the water molecules plus cation is now greater than the 60 Å which might fit into a cage formed by a dodecahedron, for example. Hence, one must conclude that the hydration is limited to two loosely held molecules.

There is a temptation to associate the phenomenon of entrainment which is so troublesome in distillation with the hydration of ions. There may, of course, be an indirect connection but entrainment must be directly due to bubble formation. The surface film tends to concentrate various types of nonvolatile impurities and to take on elastic proportions.

SUMMARY

The explanation of the unique properties of ice and water is to be found in the hydrogen-bonded structure common to both physical states. The more essential peculiar abnormalities of water are:

1. Density
2. Specific heat
3. The formation of gas hydrates
4. The high mobility of the hydrogen and hydroxyl ions.

An adequate explanation has been given for these abnormalities. Much yet remains to be explained in regard to other physical properties of water and more especially ice. A study of the viscosity and its relation to the dielectric properties should be rewarding. It should be noted that the gas hydrates are essentially a cubic form of ice.

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ION EXCHANGE MATERIALS

BY A. S. BEHRMAN*

When Dr. Buswell asked me to give this talk on ion exchange materials, I asked him how long he wanted the talk to be. He replied, "One hour." I told him I was a firm believer in the statement of Henry Ward Beecher, the great preacher, that no soul is saved after the first twenty minutes. Even so, I had to agree with Dr. Buswell—and I hope you will agree also—that an hour is all too short for an adequate treatment of just about the most fascinating chemical field I know.

It seems to me that we have two principal possible approaches to our subject. One might be called an analytical approach—that is, we could state and examine the essential features of our present-day knowledge of ion exchange materials and ways of using them in industry, particularly in water treatment.

The other approach is an essentially historical one. Here we could trace the progress of the ion exchange concept and of ion exchange materials from their earliest beginnings on to and through their present relatively high state of development.

I shall use, if I may, a combination of these two approaches. I think that we will obtain a better understanding of the ion exchange concept and of ion exchange materials by tracing their progress historically. We will then be in a more advantageous position to examine rather critically our present-day knowledge and lack of knowledge, and possibly to suggest other avenues for exploration in the future.

Ion exchange, like many other industrial chemical processes, has achieved a high state of practical utility without a completely satisfactory theoretical picture of its basic mechanism. Ion exchange is unique, however, in that, something like Venus, it sprang so well-grown at birth. Then, for almost a half century, it grew little or none. In the next half century, just completed, its progress was remarkable, though it came in periodic waves rather than as a slow and steady growth. The present intense interest and activity in ion exchange appears to be a consolidation of recent accumulations of knowledge, perhaps to be followed by another of the spectacular developments which have characterized the history of ion exchange from the start.

The origin of the ion exchange concept, as is now fairly well known, dates back a little over a century. In 1850 and 1852, J. Thomas Way, Consulting Chemist to the Royal Agricultural Society of England, presented two papers before the Society on the "Power of Soils to Absorb Manure" (22).** These two papers, totaling 86 pages, have never ceased to fascinate me. They should be required reading for every graduate student in chemistry.

Way's work was inspired by two enterprising agriculturist friends of his, Mr. Huxtable and Mr. Thompson. Mr. Huxtable stated that "he had made an experiment in the filtration of liquid manure through a bed of an ordinary loamy soil; and that after its passage through the filter bed, the urine was found to be deprived of colour and smell—in fact that it went in manure and came out water."

Mr. Thompson (21) "had found that soils have the faculty of separating ammonia from its solution: a fact appearing still more extraordinary, inasmuch as there is no ordinary form of combination by which we could conceive ammonia to be combined in a state of insolubility in the soil." Also, but apparently unknown to Way, Thompson had treated soil with a solution of ammonium sulfate, and had found calcium sulfate in the effluent.

I would like to take you step by step through Way's researches, and let you share the successive exaltations and depressions he must have experienced in a really amazing pursuit of his goal. Neither time nor, I suspect, your patience would permit. Suffice it to say, therefore, that Way first considered and abandoned the idea that the absorption of ammonia was due simply to a porous structure; other porous materials he tested did not have this ability.

By further experimentation he found that this "power of the soil to combine with ammonia is greatly diminished by burning it." This loss of combining power was not due to the destruction of organic matter; activity remained if the organic matter in the soil was burned with nitric acid.

Then came his important experiments, quite independent of those of Thompson, in which he treated the soil with solutions of ammonium sulfate, and found in the filtrate "an abundance of sulfuric acid in combination with lime."

Way postulated and tried in turn the idea that the effective agent in the soil was a lime compound; a silicate; a calcium silicate; a double silicate, such as feldspar, the double silicate of alumina and potash; and albite, the double silicate of alumina and soda; and, in turn, all results were negative.

He apparently had an inner conviction, never-

*Consultant, Chicago, Illinois.

**Number in parentheses refers to number in BIBLIOGRAPHY at end of this article.

theless, that he was on the right track. In his own words, "it was still possible, however, that these double silicates, when formed artificially by precipitation, might be capable of effecting that which in the mineral state they were unable to accomplish." He prepared such artificial double silicates by reacting sodium aluminate with sodium silicate, and aluminum sulfate with sodium silicate. The products had the long-sought property. The first synthetic base exchange materials had been produced; and it will be noted that the reagents used in their production are still being employed for the same purpose.

Way believed there was a preferential and nonreversible order in the exchange of bases. He drew up a list of this sort in the manner of the electromotive series of metals. This error was corrected a few years later by Eichhorn (10). He demonstrated the mutual replaceability of the bases. He also showed that the natural zeolites, which are hydrated double silicates, do possess base exchange properties, and that Way's negative results with natural double silicates were due to his use of anhydrous materials.

One would think that this surprisingly complete work of Way and Eichhorn would have found practical utilization fairly promptly. It did not. On the contrary, it seems to have been completely ignored for almost half a century. In 1896 Harms (14) obtained a patent covering the process of replacing potassium compounds in beet sugar juices with the less melassigenic calcium compounds, thus increasing the yield of crystallizable sugar. The base exchange material utilized by Harms was a bolus, or siliceous earth. The process was not commercially successful.

Rümpler (18), also working with the replacement of potash in beet sugar juices, demonstrated the base exchange power of cement. He also utilized a synthetic base exchange material which he precipitated in a porous medium such as charcoal. Rümpler pointed out that if these "zeolites" were regenerated with sodium compounds instead of calcium compounds, they would extract calcium, magnesium and potassium from solution.

Although Rümpler tried to lay the basis for claiming that he first suggested that zeolites could be used for water treatment, there can be little question that the real father of zeolite water softening was Robert Gans (11). It was Gans who made the first fused base-exchanging sodium aluminum silicates. He christened them "Permutits"—a name which later competitors found difficult to dislodge as a generic one.

It was on Gans' inventions that the German zeolite water softening industry was founded in the first decade of the twentieth century. Soon afterwards, as was to be expected, the Permutits were brought to the United States, and the American Permutit Company was founded.

Gans' Permutits were highly porous, hydrated sodium aluminum silicates with compositions of the anhydrous portions ranging typically from about $Na_2O.Al_2O_3.4SiO_2$ to $Na_2O.Al_2O_3.8SiO_2$. Gans' products and practice were sounder than some of his theories behind them. He drew a long and laborious distinction between "aluminate silicates" and "double silicates." In the "aluminate silicates" the alkalies and alkaline earths were supposed to be combined principally with the alumina, while in the "double silicates" the alkalies and alkaline earths were supposed to be combined principally with the silica. By the same token, effective base exchange products could be prepared by wet methods from sodium silicate and sodium aluminate, but not from sodium silicate and aluminum sulfate—the reaction product in the latter case would be a "double silicate." I was always greatly interested in this point of view, because, twenty years later, the company with which I was associated for many years developed a group of very effective gel zeolites made essentially from sodium silicate and aluminum sulfate.

The new method of water softening captured the fancy of the water treating world and took it by storm—although time was required to demonstrate that the new process really worked and was not a trick or a fraud. The incredulity was understandable.

The great success of Gans' Permutits naturally invited competition. This competition was facilitated by the fact that in the United States, unlike in Germany, no broad process patent was obtained on the softening of water with base exchange silicates. It is true that the American Permutit Company was able to maintain for several years an almost complete monopoly on the zeolite water softener business by asserting in the courts that a certain Gans apparatus patent (12) covered all downflow softeners. The lengthy and expensive litigation finally reached the Supreme Court, which invalidated this interpretation of the patent.

Before this litigation was well started, however, two important competitors of Gans' fused zeolites had already made good headway. The more important of these was processed glauconite, or greensand, first sold as the baked product under the name of "Borromite," after the inventor, George Borrowman, of Chicago. Glauconite, as you all know, is a granular material of marine origin found rather extensively in portions of the Atlantic Seaboard. The most important deposits, from the water softening standpoint, have been found in New Jersey. This material is essentially a hydrated potassium iron silicate.

Greensand had several important advantages over Gans' fused zeolites. The raw material was relatively cheap and it was easily processed. Even

more important were its operating characteristics. It was very rapid-reacting, both in softening and in regeneration—quite in contrast to Gans' fused zeolites, which could be used at only slow rates during the softening cycle, and which were typically regenerated overnight, or, if a spare softening unit was provided, during an entire working day. Greensand was also economical in the amount of salt required for regeneration—low enough in fact, that the "salt-saving" systems used with the slow-acting, high salt-consuming fused Permutits were largely abandoned. The two disadvantages of greensand were its low total exchange capacity per cubic foot—only a half or third that of the fused Permutits—and its small size and spherical shape, which tended to make it pack and cause serious pressure losses in long runs.

Almost concurrently with greensand appeared Refinite, a baked and rehydrated weathered bentonitic clay found in Nebraska.

In the meantime, synthetic siliceous base exchange materials prepared by wet methods had begun to appear. In 1913 Bochringer and Gessler (6) obtained a patent for precipitating a sodium aluminum silicate from dilute solutions of sodium aluminate and sodium silicate. The flocculent gelatinous precipitate was filter-pressed, washed, dried, and granulated. The product, known as "Decalso," was produced and sold by the American Zeolite Company, later taken over by the Permutit Company.

The really noteworthy development, however, in synthetic base exchange silicates came in the 1920's with the announcement of the "gel zeolites." Working quite independently, Wheaton (23) in England produced a gel zeolite from sodium silicate and sodium aluminate, and Behrman (5) in the United States prepared the first of his gel zeolites made from sodium silicate and aluminum sulfate. Gel zeolites are produced by reacting solutions of the proper concentrations in the right proportions, at the proper temperature, and with the proper method of mixing. Under these optimum conditions no precipitate results; instead, the entire reaction mixture sets to a firm stiff hydrogel. This colloid system is irreversible. When the hydrogel is dried, it forms hard, insoluble, highly porous particles possessing the properties of high exchange capacity, rapid rate of exchange in softening and regeneration, and low salt consumption.

The Wheaton gel zeolites were produced in the United States under the trade name "Doucil" by the American Doucil Co., in which the Philadelphia Quartz Co. was interested. The Behrman gel zeolites were produced by the International Filter Co. (now Infilco Inc.), under the trade name "Crytalite."

The characteristics of the gel zeolites made them particularly suitable for household softeners, and for many—but not all—types of industrial and municipal softeners, especially for very hard waters. Their chief disadvantages were—and are—their relative lack of stability and their loss of silica under some conditions, particularly high temperatures and either unusually high or low pH. Competition for the Wheaton and Behrman gel zeolites developed shortly. Of greatest interest in connection with these competitive materials was the method of drying the hydrogel out-of-doors, as practiced, for example, by the National Aluminate Corporation and the Culligan Zeolite Company, and the method of drying by freezing as practiced by the Research Products Corporation. Production by natural drying out-of-doors has at times reached very large proportions; drying of gels by freezing has had, to the best of our knowledge, only a limited application.

To make this greatly condensed history of siliceous base exchange materials a little more complete, it should be mentioned that patent litigation was not completely absent during this period of technical and commercial development.

We have now come to the early "thirties." I think it is worth pausing here just a moment to survey the situation as it was at that time. By then zeolite water softening was the generally accepted method of treatment wherever "zero hardness" was the first consideration. Accordingly, a zeolite softener was fast becoming standard equipment for laundries, for many textile dyeing and finishing plants, and for a host of other uses where lack of any hardness in the water was a proper criterion. Because of pressure tank design and convenience in operation, household softeners began to be "musts" in better homes in hard water areas.

By and large, the zeolite of choice in the early thirties was greensand for boiler feed and for most other industrial applications where the raw water was relatively soft or only moderately hard, and the gel zeolites for household softeners and for industrial and municipal installations when the raw water was quite hard.

Not that there was general acceptance of zeolite softening for boiler feed. The first flush of enthusiasm was over, and there was growing appreciation of the possibilities of foaming, priming, and corrosion when using for boiler feed a water of appreciably high bicarbonate hardness softened by zeolite. Zeolite followed by acid treatment and aeration was being used in a number of instances to try to overcome these limitations.

Then came the "thirties." With them began what we might call the modern age of ion exchange—an age in which we, of course, are still living.

The first herald of this modern age was the appearance of organic base exchange materials. These were the sulfonated coals, prepared by the action of concentrated or fuming sulfuric acid or sulfur trioxide on bituminous or anthracite coal. With what has always seemed to me to be a striking

lack of originality, these new materials were dubbed "carbonaceous zeolites." Another attempt to link the future somewhat more broadly with the past was the suggested name "organolite"; but this hybrid never received more than very limited and temporary acceptance.

The "carbonaceous zeolites" marked two important advances in water treatment. One was that they were non-siliceous, and so could be used for softening boiler feed water without danger of contributing to the formation of silicate scale—a danger to which the boiler water field was becoming acutely sensitive. Furthermore, these "carbonaceous zeolites" had two or three times the exchange capacity of the same volume of greensand, they were rapid-acting, efficient in salt consumption, and rugged.

The other, and far more important advantage, however, was the fact that the "carbonaceous zeolites" could be used as hydrogen exchange materials by regeneration with an acid instead of a salt. The idea of a "hydrogen zeolite" was not new; but it had not been possible to practice the idea industrially before, simply because the siliceous base exchange materials, even the most rugged of them, could not stand up under repeated regenerations with the strong acid solutions required.

It is not generally appreciated that organic cation exchangers were known earlier. In 1922 Hepburn (15), an Englishman, was granted a patent on softening water using peat as the base exchange medium. While theoretically possible, the method was obviously not very practical or attractive. Borrowman (7) went a step further, in his patent issued in 1931. He used lignite. It is perfectly feasible to soften water by passing it through a bed of lignite particles, and subsequently regenerate the exhausted bed in the usual way with common salt. The principal deficiency of lignite is its low exchange capacity. At best this is only about two-thirds that of greensand, which, as we have seen, was already losing ground because its capacity was so low. It was doubtless for this reason that Borrowman's lignite never achieved any commercial importance.

In 1940 a group of United States basic patents was granted on sulfonated carbonaceous materials to Liebknecht (16) of Germany, and Smit (19) of Holland. These United States patents were all assigned to or otherwise controlled by the American Permutit Company. It is interesting to note that during the prosecution of the patent applications in the Patent Office the Examiner cited a British patent to Halse (13). This patent disclosed a method for purifying beet sugar juices by means of a "charcoal" prepared by the action of sulfuric acid on lignite, wood, and various forms of cellulose. Halse said that this product had all the advantages of animal charcoal, and in addition would remove a large proportion of the salts present. It could be revivified by treatment with sulfuric or hydrochloric acid.

The first practical application of hydrogen exchange in water treatment in the United States of which I am aware was for the manufacture of raw water ice (2). The principal impurity in the water in this case was sodium bicarbonate. By hydrogen exchange, using a sulfonated coal regenerated with sulfuric acid, the sodium bicarbonate was converted to water and carbon dioxide. In other words, if a raw water contains only bicarbonates, the dissolved solids content can be reduced by hydrogen exchange to zero.

The "carbonaceous zeolites" had only a few years to become firmly established in industrial water treatment when the real revolution in ion exchange materials and methods took place. This was the classic paper by Adams and Holmes (1) in 1935 announcing in one fell swoop (1) purely synthetic cation-exchange resins, (2) anion exchange, and (3) anion-exchange resins.

The cation-exchange resins of Adams and Holmes were interesting, but not startling, as far as their immediate impact on the water treatment field was concerned. After all, their original resins, prepared by reacting phenolic compounds and formaldehyde, had very little to offer that the sulfonated coal materials had not already made available.

The real sensation, of course, was the demonstration of anion exchange, by synthetic resins prepared by reacting certain types of amines and an aldehyde. In this paper, Adams and Holmes suggested and demonstrated the obvious corollary of their work, that is, the preparation of "de-ionized" or "demineralized" water by a combination of hydrogen exchange and anion exchange.

One would have thought that Adams and Holmes would have obtained very broad process and product patents covering their pioneering achievements. They did not do so, at least in the United States. Even before the Adams and Holmes patents were issued in the United States, several forward-looking companies had made such good progress in developing competitive materials that a number of "de-ionizer" installations had been made.

The first anion exchange resins placed on the American market were not satisfactory. These were a m-phenylenediamine-formaldehyde resin and "aniline black." A common characteristic of these two materials was that they would lose exchange capacity rather rapidly, and without losing volume. In consequence, replacement of a mass of inactive material was required at relatively short intervals.

I believe that the first anion-exchange resin with reasonably satisfactory capacity and operating characteristics was a guanidine-aldehyde resin known as "Anex 299," developed by International Filter Company, now Infilco Incorporated. This resin was used in a number of water-demineralizing installations and in the first full-scale sugar juice

treatment plant, in combination with "Catex," a sulfonated coal hydrogen exchanger, and an intermediate activated carbon bed. This installation was made in 1941. ("Anex" and "Catex" are Infilco's generic trade names for the anion and cation exchangers they produce or sell.)

The original "Anex" was a very useful resin, and for several years found fairly extensive application. It was gradually superseded, however, by newer types of anion-exchange resins possessing considerably higher exchange capacity and greater stability. Among these may be mentioned the "Duolites," produced by the Chemical Process Company; the anion-exchange "Amberlites," of Rohm and Haas; the "Ionac" anion resins of American Cyanamid; and the newer "Deacidite" resins of Permutit.

At this point in our historical sketch it is obvious that we have approached so closely to the present day that we are dealing with practically current history. Accordingly, let us endeavor to summarize briefly the important developments of the past ten years, both in ion exchange materials and in novel methods of using them in water treatment.

First, with respect to cation exchange materials, we have the following principal categories:

1. The inorganic gel zeolites, the hydrated sodium aluminum silicates we have previously discussed in detail. Substantial quantities of these materials are still being produced, especially for household softeners. Their use is steadily declining, however, in favor of organic high-capacity* materials.

2. Greensand. This low cost, low capacity inorganic material is still being used, particularly on low hardness waters; but its low capacity and siliceous nature are contributing to its increasing abandonment in favor of efficient organic materials.

*The exchange capacities of both cation and anion exchangers are affected very appreciably by a number of factors, such as depth of bed and particle size of the exchanger, composition of the water being treated, rate of flow of the water through the bed, and the amount of salt, acid, or alkali used in regeneration. This fact is recognized and utilized in the design and operation of ion exchange plants. Accordingly, it is not possible to state precisely the capacity of any ion exchanger without specifying the conditions of use. The manufacturer or supplier should be consulted in each case. For purposes of approximate orientation, however, the following capacity ranges will be found helpful; all capacities are expressed in terms of grains of calcium carbonate equivalent per operating cubic foot of the exchanger: Greensand, 2,500-3,000; gel zeolites, 8,000-12,000; sulfonated coals, 7,000-10,000; cation exchange resins (a) low capacity, 7,000-10,000, (b) medium capacity, 12,000-15,000, (c) high capacity, 25,000-35,000; anion exchange resins (a) medium capacity 15,000-20,000, (b) high capacity, 25,000-35,000.

3. Sulfonated coals. These materials have replaced greensand in many industrial installations. They also have a ruggedness and certain other operating characteristics which enable them to compete — for the present, at least—with synthetic cation exchange resins of higher capacity.

4. Synthetic resins. This is a category which we must break down into three principal subdivisions:

(a) Phenol formaldehyde, sulfonated formaldehyde and similar types of resins. The presently produced resins of this type constitute a group of medium exchange capacity (characteristically a little higher than that of the sulfonated coals). They are useful both for sodium exchange in water softening and in demineralizing and other industrial work.

(b) The high capacity "nuclear sulfonated hydrocarbon polymers," more specifically, the sulfonated polystyrene resins. Several manufacturers are producing resins of this type. All of them, I believe, are licensed under the General Electric Company's patent issued to D'Alelio (9). These resins are becoming increasingly important in the cation exchange field. Their capacity is two or three times that of the medium capacity synthetics, and roughly ten times that of greensand. They are replacing to a considerable extent the gel zeolites household water softeners, primarily because of their higher capacity (about twice that of the gel zeolites) and because of their economical salt consumption. They are rugged and long-lived. Their ability to withstand high temperatures is opening up a new type of hot process softening which we will discuss in more detail very shortly. The principal present limitation appears to be a high acid consumption in the hydrogen cycle. It would not be surprising if this characteristic was improved. This type of resin is characteristically supplied in bead form.

(c) The carboxylic type cation exchange resin. This resin is unique in that being essentially a solid weak acid, it will not react appreciably in the hydrogen cycle with neutral salts of strong acids, as for example, sodium chloride. It is therefore especially useful in water treatment for removing bicarbonate alkalinity without seriously affecting the neutral salt concentration.

There are a number of modifications of the cation exchange materials mentioned above, but I do not believe that they warrant discussion here. Neither do I think we need deal here with processed clay zeolites or with the so-called "super greensands," because of their relatively limited use.

When we come to anion exchange materials, the products on the market are all synthetic resins. In all cases, as far as I know, the activity derives from basic nitrogen groups.

Here again we have three principal classes,

with numerous modifications and variants:

1. Medium capacity resins, such as the guanidine-melamine-formaldehyde type.

2. High capacity, weakly basic resins, consisting principally of polyamine-aldehyde condensation products. These resins may be considered to be solid weak bases. They are very effective for removing strong acids such as sulfuric and hydrochloric, but only slightly effective with weak acids such as carbonic and silicic. These resins are the ones most commonly used in the ordinary deionization plant.

3. Strongly basic resins. Within the last two or three years these quaternarized materials have become available. They are industrially important because they are very effective for removing weak acids. This makes them particularly valuable for the direct removal of silica from water, a method which is more generally desirable than the ingenious but indirect method of converting the silica to hydrofluosilicic acid and removing this with one of the weakly basic resins previously available. Since the new strongly basic resins have relatively low capacity for the strong acids, they are usually employed for removing silica or carbon dioxide in a separate step following the conventional cation and anion exchange treatment; but in some cases, especially where the total anion load is not too great, they may be used in a conventional two-bed system.

How then can we summarize briefly our present position in the cation and anion exchange material picture, particularly with respect to water treatment? As regards cation exchangers, the situation is good. We now have a whole range of materials from low capacity greensand through sulfonated coals, medium capacity resins, gel zeolites, carboxylic resins, and high capacity, thermally stable sulfonated polystyrenes. All of these materials are presently available at reasonable prices; and the prices of some of the newest materials will probably be reduced when patents expire or demand increases.

The situation in anion exchangers is not so good. It is true that we now have weakly basic anion exchange resins with capacities comparable to those of the high capacity cation exchangers; and we have quaternarized strongly basic resins especially useful for removing silica. But no anion exchange resin that I know of has the long life and ruggedness of the newer cation exchangers; none can be used at high temperatures; and the price of all of them is high. This high price, however, is admittedly not nearly as important in water treatment as in some other industrial applications where the resin is consumed rather rapidly. There is still a great deal of room for improvement in anion exchange materials to remedy the deficiencies just mentioned. When we have anion exchange materials comparable in operating characteristics and price with the high capacity cation exchangers now available, we will have reached an attractive plateau in water treatment.

I have to take issue with Bauman's statement (3) in his excellent chapter in which it is found that "the [ion exchange] resin is expected to serve for a period of 25-50 years in constant contact with the solvent and with no loss in weight and no physical or chemical degradation . . . ." I don't know of anyone in the operating end of the ion exchange field who really cherishes this exceedingly great expectation. For many years the producers of ion exchange materials, and the manufacturers of equipment in which they were used, penalized themselves entirely needlessly by preaching that the ion exchangers would last indefinitely. They do not. Ion exchanger replacement is as necessary and inescapable an operating cost as are the chemicals and water used for regeneration. It happens that in water treatment the unit replacement cost is relatively small, but it is there. The equipment manufacturers finally awoke to the unsoundness of their own pronouncements. The most common practice now is to guarantee the ion exchanger either (a) for a specified volume of treated water or (b) against more than a specified percentage annual loss for a few years (commonly 5 per cent per year for 3 years in the case of the stable cation exchangers).

What of the future? Speculation of this sort is always hazardous, but frequently, as in this case, very intriguing.

What will probably happen first will be a continuation of what is happening right now, that is, a greater understanding and appreciation of the ion exchange resins we now have, and a more effective use of their unique properties. Two fairly recent examples are in point.

One is the utilization of the thermal stability of the sulfonated polystyrenes in a hot-process combination lime and cation exchange softening for boiler feed water. This process, which combines the merits of the two types of treatment and overcomes their limitations, promises to be one of the most important advances in boiler feed water treatment in many years.

The other example utilizes the salt-splitting properties of the strongly basic anion exchange resins. This suggested method, still in the experimental stage, replaces the bicarbonate alkalinity in the raw water with the relatively inocuous sodium chloride. The effective agent is a bed of strongly basic anion exchange resin in the chloride form, produced by regenerating with sodium chloride. A second bed of high capacity cation exchange resin in the sodium form replaces calcium and magnesium with sodium. The two beds may be in the same tank or in separate tanks. Sodium chloride is the regenerant for both beds.

An obvious fundamental objection to the second example just cited is that the process is purely

one of replacement, not removal. At the present stage of experimentation, the salt consumption is also reported to be high. Both objections—assuming that the salt consumption cannot be improved—would limit the utilization of the method, in spite of its attractive simplicity. I have cited this example primarily to illustrate the property of salt splitting, which I believe will be made use of more extensively in the future.

I think that greater use will be made also of the color-removal properties of the ion exchangers in water treatment. It is not generally appreciated that certain of the ion exchangers, both cation and anion exchangers, have marked color-removal ability which is actually being used in the treatment of other liquids. The color-removal property depends, of course, not only on the ion exchanger but on the nature and charge of the color body particle. Several American resin producers are now turning out resins designed primarily as adsorbents, not ion exchangers; and at least one such material (a low capacity anion exchange resin) has been used successfully in Europe for several years in the decolorization of corn sugar liquors.

And it would not be surprising if some quite new cation and anion exchangers appeared in due course. My guess is that these will not come until after we have a better fundamental understanding of ion exchange and ion exchange materials than we do now. Our theory is just beginning to catch up with a century of remarkable empirical progress. After our theory does catch up, and after we begin to think inductively on the basis of a sound theoretical explanation of what has been done in the past, we may see entirely new vistas and horizons.

In this present period of trying to find the right theory to fit the data, I think sometimes that we are so anxious to fit the data into our more familiar physico-chemical theories and terminology that we lose the forest for the trees. I would like, if I may, to give you the same elementary picture of ion-exchange that I gave last week in Chicago during lunch with a patent attorney friend who is also a chemist. He asked me point blank to tell him in plain words how ion exchange works. I told him that I didn't know, but that I would try to give him a simple picture that I had found very helpful to my own thinking: "Any ion exchange compound may be regarded as a matrix or as a clamp holding an ion by adsorptive forces of a given magnitude. In the case of a cation exchanger, the ion clamp is acidic; in the case of an anion exchanger, the ion clamp is basic. The adsorbed ion can be knocked out and replaced by a different ion under one of two circumstances. In one, the new ion is adsorbed preferentially, that is, there is a greater inherent attraction or 'affinity' between it and the adsorbent. In the second case, the attraction or 'affinity' of the new ion is equal to or less than that of the ion first adsorbed, in which case the adsorbed ion is pushed out by the sheer mass effect or 'mass action' of an excess of the new ions. If this second possibility were not true, we could not have reversibility of ion exchange. Ion exchange materials, like most other adsorbents, must be highly porous, to provide the large amount of surface and surface attractive forces required. An ion exchange material is a particular kind of an adsorbent, with a mobile adsorbate. Like any other adsorbent used with aqueous solutions, it must be insoluble in water."

I would like to pursue that line of thought a little further. The first practical ion exchangers, as we have seen, were the sodium alumino-silicates. Here we can picture our adsorbent, or ion clamp, as a highly porous alumino-silicic acid. An interesting fact is that silicic acid itself, in the form of silica gel, is an ion exchanger, though not a very good one. Wheaton (24) describes the preparation of a cation exchanger by the partial neutralization of a sodium silicate solution with an acid. The resulting hydrosol sets in due course to a stiff hydrogel, which is dried, washed, and granulated. The product may be regarded as essentially a highly porous silica gel containing a very appreciable quantity of adsorbed sodium ions. It may actually be used to soften water, and, when exhausted, it may be regenerated with sodium chloride. The initial exchange capacity is quite low, however, and diminishes with subsequent regenerations.

If, however, the adsorbent is not pure silicic acid, but an alumino-silicic acid, both the exchange capacity and the stability of the ion exchanger are improved. This effect reaches a practical maximum when the ratio of Al_2O_3 to SiO_2 is about 1:6 to 1:8; and in the manufacture of gel zeolites, the proportions of sodium silicate and sodium aluminate or aluminum sulfate are usually chosen which will provide an Al_2O_3:SiO_2 ratio in the range just indicated.

Following the same line of thought with inorganic anion exchangers, I would like to point out merely that the highly porous gels of the oxides or hydroxides of several of the heavy metals have very appreciable anion exchange capacities (17).

With respect to the ion exchange resins, an excellent discussion of their structure and properties has been given by Bauman (4) and will not be repeated or elaborated on here. It is heartily recommended to anyone interested in the ion exchange field.

What about future physical methods of employing ion exchange materials? I think we are beginning to see already what the trend will be. The mixed bed of cation and anion exchangers offers many interesting possibilities. The problem here, of course, is regeneration. The "monobed" system of Rohm & Haas is an ingenious solution. It is interesting to note that a recently issued United States patent (20), based on an application in Holland

in 1939 claims the purification of an impure sugar solution by contacting it with a mixed bed of organic cation and anion exchangers.

Another indication of the future trend is the announcement by one of the equipment manufacturers of a continuous process for either cation or anion exchange. Again, there is the recent patent (8) covering a continuous system in which the liquid is treated with a suspension of mixed cation and anion exchangers; the treated liquid is separated from the mixed exchangers; the cation exchangers are separated from the anion exchangers through differences in their physical properties, such as density, size, shape, etc.; the separated exchangers are continuously regenerated and returned to the treating system.

Since I have long had a decided liking for continuous processes of any sort, wherever they can be justified, this trend in ion exchange is very heartening. Possibly I should say, a return to this trend, since the idea of continuous systems in ion exchange is anything but new. The question of justification in any particular case is almost always one of simple economic balance: It would obviously be difficult to find justification for a continuous household softener or other similar installation where a small capital outlay provides for an uninterrupted supply of soft water for a week or more, and where the cost of the labor and salt for regeneration are unimportant. At the other extreme, however, where conventional installations are very large, and cycles short, a continuous system may make possible important economies in both capital outlay and operating expense.

Before I close, I would like to pay a well-deserved tribute to the producers of ion exchange materials and to the manufacturers of ion exchange equipment. They and their research, development, and sales organizations have played a major role in carrying the gospel of the ion exchange idea and its almost unlimited potential applications.

And so we come to the end of our discussion of what to me are the really significant aspects of ion exchange materials and methods of using them in water treatment. I have not felt warranted in taking time to discuss the special silver zeolite and auxiliary chemicals employed to de-salt small quantities of sea water for downed aviators or shipwrecked mariners. Nor have I felt justified in describing the magic of ion exchange in its manifold non-water applications, which now run the entire gamut from sugar juice purification, to the separation of nuclear fission products, to the recovery of the antibiotic "wonder drugs," to the relief of acidosis. The end is not even remotely in sight.

And it is worth remembering that it was all started a century ago when J. Thomas Way found out what it was in soil that, in Mr. Huxtable's words, converted liquid manure to water.

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BOILER FEED WATER CONDITIONING

BY FREDERICK G. STRAUB*

"If any apology is due for discussing the somewhat perennial subject of boiler water it might be fair to say, perhaps, that the matter is coming to be treated in a more rational manner; that much of the quackery and mysticism that has so long prevailed is being superseded by a more scientific and sensible consideration of the problem. The study of some phases of the subject have been under more or less continuous investigation in the department of Applied Chemistry for the past two years---." Professor S. W. Parr made this statement in an article on Boiler Waters which appeared in The Technograph, a publication of the Association of Engineering Societies of the University of Illinois, volume No. XI, 1896. Professor S. W. Parr continued his research on boiler waters for the remainder of his life. In 1924, the author returned to the University of Illinois and worked with Professor Parr on boiler water research. Boiler water research is still being conducted at the University of Illinois, and we may now modify Professor Parr's statement and say, "The study of some phases of the subject of Boiler Water Treatment has been under more or less continuous investigation in the department of Chemical Engineering at the University of Illinois for the past fifty-seven years. The last twenty-seven years of the research has been carried on in cooperation with the Utilities Research Commission of Chicago."

Boiler feed water conditioning covers the treatment of the waters used in the steam-water cycle of the steam power plant. The cycle may be very simple, as in a small steam heating plant, or it may be quite complex when electric power generation and the use of process steam are involved, as in a large chemical manufacturing plant. Figure 61 is a schematic diagram of a power plant showing types of difficulties which may be encountered in the steam-water cycle. These difficulties may be classified under the following headings:

I. Scale
II. Corrosion
III. Steam contamination
IV. Embrittlement

Any attempt to cover the entire field of boiler water treatment in such a short period of time would be futile; consequently, only one or two phases of the subject will be discussed with the purpose of showing how the various laws of physics and chemistry have been applied to these problems. The problem of steam contamination as it applies to deposits on steam turbine blades serves as an excellent example of the application of physical chemistry principles in solving the problem.

Two types of deposits are found on the turbine blades. The deposits are either soluble or insoluble in water. The soluble deposits from near the point where high pressure superheated steam enters the turbine and are found on the blades in the high pressure portion of the machine. The deposit dissolves in hot water and is made up of the materials normally found in the boiler water, but not in the same ratio as they occur in the boiler. The major constituent of the deposit is sodium hydroxide with small amounts of chlorides, sulfates, silicates, and iron oxides also present. The deposits occur in turbines operating in all pressure ranges, but only in turbines using superheated steam.

Deposits occur even when steam containing very small amounts of dissolved solids is used. Thus, in one large plant having steam with a specific conductance of one micromho per cm.^{3} (specific resistance of one million ohms per cm.^{3}) it was necessary to take the load off the large turbine and remove the deposits by means of washing the turbine with wet steam every four weeks. The total dissolved solids in the steam was less than 0.5 ppm. Since this turbine used about a million pounds of steam per hour, it meant that about twelve pounds of solids passed through the turbine in twenty-four hours.

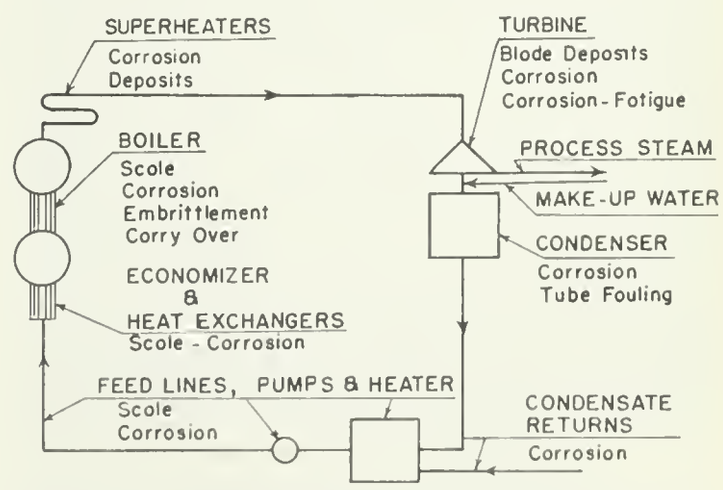


FIG. 61. —SCHEMATIC DIAGRAM OF POWER PLANT, SHOWING TYPES OF DIFFICULTIES WHICH MIGHT BE ENCOUNTERED.

*Research Professor of Chemical Engineering, University of Illinois, Urbana, Illinois.

If even a small amount of this adhered to the blades, it would reduce the load-carrying ability of the turbine after a short period of operation.

In order to determine whether there is present in the average boiler water any material which would cause adherence or sticking to the turbine blade, it is advisable to study the behavior of the various salts encountered in boiler waters as they pass from solution in wet steam to superheated steam. When only pure water is present in the boiler and a small amount of the boiler water is mechanically carried into the steam and to the superheater, the droplets of water will vaporize in the superheaters and no free moisture will be present in the superheated steam. However, if sodium chloride, sodium carbonate or sodium sulphate is present in the boiler, the droplet of water entering the superheater with the steam will contain a dilute solution of the salts. As the steam becomes superheated, the water will vaporize and leave dry powdered salts present with the superheated steam. These salts all have fusion points well above the temperature encountered in superheated steam; consequently, they will pass through the steam pipes and turbine as a fine powder or dust and cause no appreciable difficulty. This is based on the assumption that there is only a small percentage of moisture in the steam.

However, if sodium hydroxide is present, the foregoing will not happen due to the fact that sodium hydroxide solutions behave differently than those of the other salts discussed. When a solution of sodium chloride, carbonate, or sulphate is boiled, the solution is concentrated as the steam is released, and the solution concentrates until a saturated solution is obtained. Any further release of steam results in the precipitation of the salt. Eventually, if the boiling is continued, all the water is evaporated, and dry salt or salts is or are left behind. If a solution of sodium hydroxide is boiled, the solution is concentrated with a continual increase in temperature until a stage is reached where the concentrated solution is in equilibrium with the vapor pressure of the surrounding atmosphere and further heating would cause a release of steam to the surrounding atmosphere, with an increase in the concentration of the caustic solution and increase in the temperature of the solution. This is shown graphically in Fig. 62, and may be illustrated as follows: If a drop of water containing sodium hydroxide in solution leaves a boiler at 600 psia. pressure and enters the superheater with the steam, the water will vaporize, thus concentrating the solution. When the temperature of the steam reaches 600 deg. F., the concentration of the caustic will be about that of a 60 per cent solution, as shown in Fig. 62, at A. If the temperature and pressure remain constant, the droplet of caustic will remain 60 per cent sodium hydroxide and 40 per cent moisture even in contact with superheated steam. As the temperature

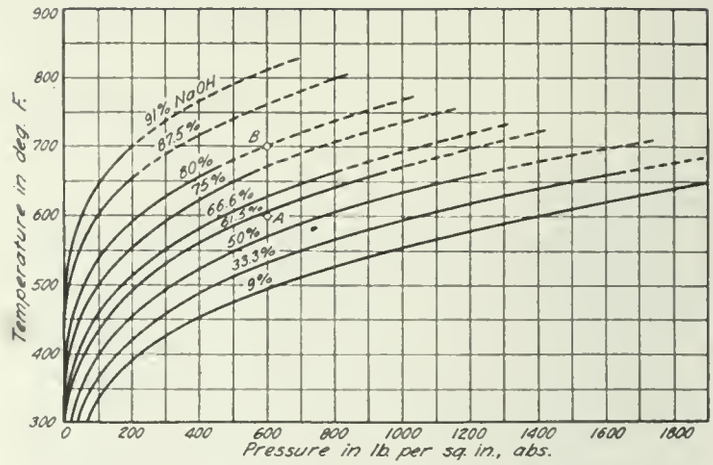


FIG. 62.—CONCENTRATION OF SODIUM HYDROXIDE IN RELATION TO TEMPERATURE AND STEAM PRESSURE.

increases with the pressure remaining constant, the caustic concentration will increase. At 600 lbs. pressure and 700 deg. F. the concentration will be as shown at point B, that of an 80 per cent solution.

The concentration of caustic in equilibrium with superheated steam at various pressures and temperatures is shown in Fig. 62, from which it is seen that the concentration of sodium hydroxide reached in the average higher-pressure plant is between 80 and 90 per cent. A solution of sodium hydroxide containing between 10 and 20 per cent moisture at these temperatures will be in a pasty or semifluid state, and in going through the turbine will adhere to the blades. The other salts, being present as a fine dust, will adhere along with the sodium hydroxide, but not necessarily in relative proportions in which they exist in the boiler water. When the temperature of the steam in the turbine is lowered until saturated steam exists, these salts will be dissolved and thus will be washed off the blades. The experience which turbine operators have had with the washing of turbines confirms this.

In order to substantiate or disprove this theory, tests were conducted in the laboratory. The procedure followed in these tests was to produce steam from a small boiler, contaminate the steam with a solution of a particular salt or combination of salts, superheat the contaminated steam, pass it through an orifice so as to impinge on a stationary blade, and then condense it at a predetermined pressure. Examination of the blade at the end of the test would serve as a measure of the amount of deposit forming. These tests showed that sodium hydroxide formed a heavy deposit while sodium chloride and sulfate did not deposit on the blade.

Mixtures of sodium sulfate and hydroxide were tested, and it was found that when the sodium sulfate was present in an amount greater than 5 times the sodium hydroxide (by weight) no deposit formed.

These results indicated that the salts which form a dry powder in the superheated steam adhere to the surface of the droplet of concentrated pasty sodium hydroxide, and, if present in sufficient amounts, entirely coats the particle and allows it to pass through the turbine without sticking to the blades. In 1934, the large power plant in which the turbine was being washed every four weeks increased the sulfate content of the boiler water so the ratio to the hydroxide was above five. During the subsequent seventeen years that this treatment has been in use, the turbine was never washed and when it has been opened for overhaul about every three or four years, it has been found free from deposits. This method of treatment has been used in many other power plants with similar results.

The insoluble deposits occurring on turbine blades are almost pure silica, either amorphous or crystalline. They form in the lower pressure portion of the machine just ahead the point where saturation of the steam occurs, but still in the area where the steam is superheated. These deposits are not water soluble; consequently, it is rather difficult to remove them once they form. The deposits occur mainly in turbines where the initial steam pressure is greater than 800 psi. The deposits of silica often form when no deposit forms in the higher pressure portion of the turbine.

It has been rather difficult to give a logical explanation of how the silica present in the boiler water as sodium silicate is transported through the high-pressure turbine blades and deposited on the low-pressure blades as crystalline silica. Splittgerber* presented experimental data which showed that silica could be vaporized from silicic acid in appreciable amounts at 100 atm. steam pressure. These results showed that the silica in the steam increased with increasing silica concentration in the boiler water. As the pressure increased, the silica in the steam increased. The addition of NaCl, Na_2SO_4, NaOH, and Na_3PO_4 to the boiler water caused the silica in the steam to decrease. The values given by Splittgerber were not readily correlated with data available from boiler operation.

If this work of Splittgerber could be carried further and additional data collected as to the feasibility of silica leaving the boiler as vaporized silicic acid, a logical explanation would be available to account for the deposition of the silica crystals in the low-pressure turbines. If silicic acid has an appreciable vapor pressure at higher steam pressures, the sodium silicate in the boiler hydrolizes, and silicic acid would be present in the steam in amounts depending upon the mole concentration of the silicic acid in the boiler water and the vapor pressure of silicic acid at that temperature. Since the concentration in boiler water of the silicates is very low, the amount in the steam would be much lower than that corresponding to the true vapor pressure of silicic acid. When superheated steam containing this amount of silicic acid is dropped in pressure and temperature, a point will be reached where the silicic acid in the steam becomes greater than the amount corresponding to the vapor pressure of solid silicic acid. When this point is reached, the silicic acid in the steam will leave the superheated steam and deposit as a hydrated silica deposit. If the steam reaches the saturation point before this occurs, the silicic acid, being slightly soluble in water, will be dissolved in the condensate and no deposit will form.

Since this appeared to be a logical approach to this problem, laboratory work parallel with power-plant tests was inaugurated in order to see if Splittgerber's method of approach was applicable to the problem of silica blade deposits. The laboratory work was conducted along two lines as follows:

1. The relationship between silica in the steam and soluble silica in the boiler water at various pressures.

2. The relationship between solid silicic acid and the silica in the steam at various pressures and temperatures.

Tests run with various amounts of sodium silicate in the boiler water at various pH values showed that the steam contained silica but that the specific conductance and pH value of the steam was constant. This indicated that the silica was not present in the steam as the sodium silicate, but was there as the silicic acid, which had very little effect on the conductance or pH value. At a given steam pressure the amount of silica in the steam was proportional

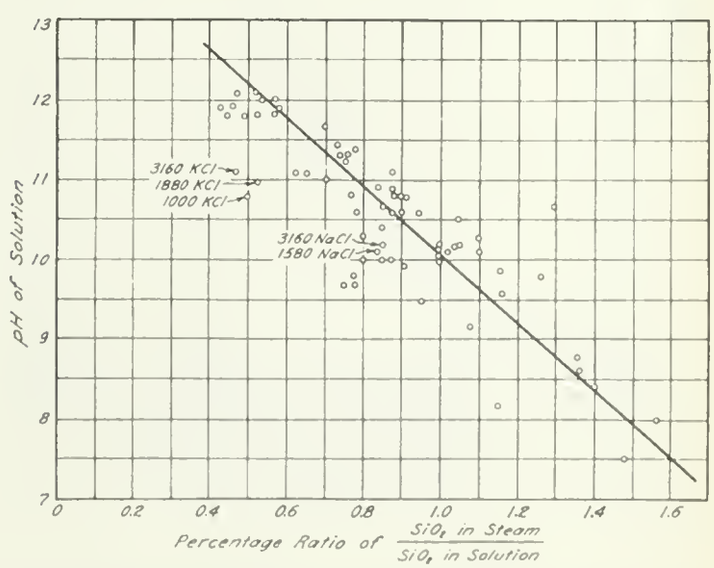


FIG. 63. —RATIO OF SILICA IN STEAM TO SILICA IN SOLUTION VERSUS pH OF SOLUTION.

*The Volatility of Silicic Acid, by A. Splittgerber, Archiv. fur Warmewritschaft, vol. 22, 1941, p. 66.

to the amount of silica in the boiler water. Then the pH of the boiler water was increased, the amount of silica in the steam decreased. Fig. 63 shows this effect of the pH of the boiler water at 1545 psi. Fig. 64 shows the effect of boiler pressure on the amount of silica found in the steam. Fig. 65 shows the silica in the steam plotted against the reciprocal of the absolute temperature, which is a typical vapor pressure curve.

Tests were conducted in which superheated steam was passed under pressure through a bomb containing solid silicic acid. Fig. 66 shows the silica concentration in the superheated steam in contact with the silicic acid versus the reciprocal of the absolute temperature and also is a typical vapor pressure curve.

The laboratory tests show that sodium silicate in boiler water at steam pressures above 700 psi. will liberate silica, presumably as silicic acid, in appreciable amounts without mechanical entrainment of boiler water. It also increases as the pH

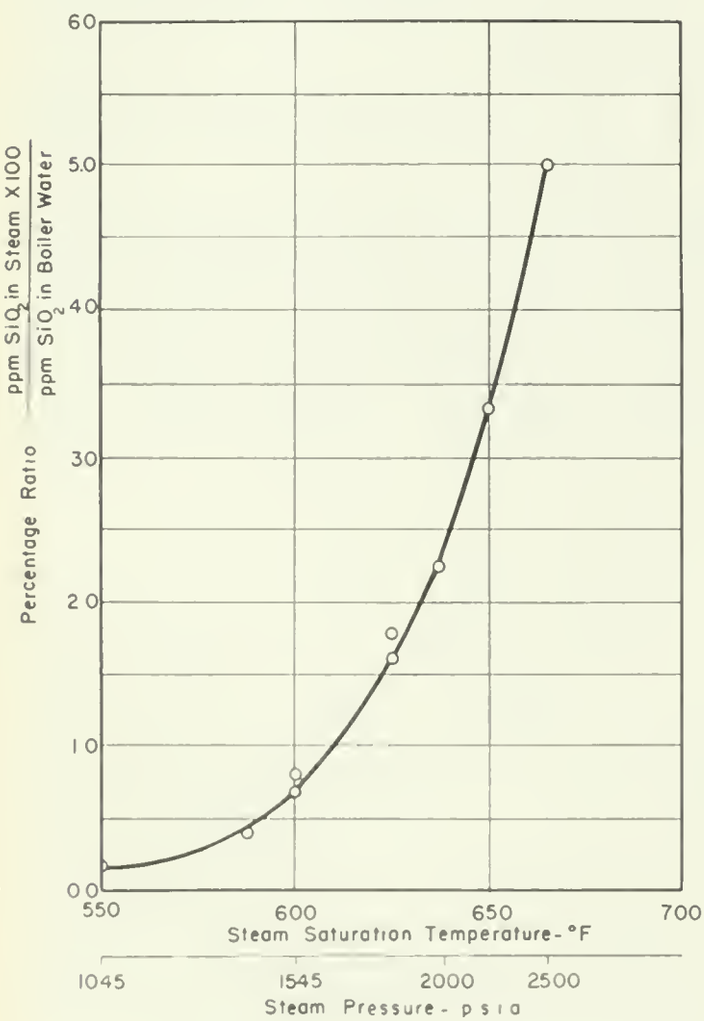


FIG. 64. —RATIO OF SiO_2 IN STEAM TO SiO_2 SOLUTION VERSUS SATURATED STEAM PRESSURE.

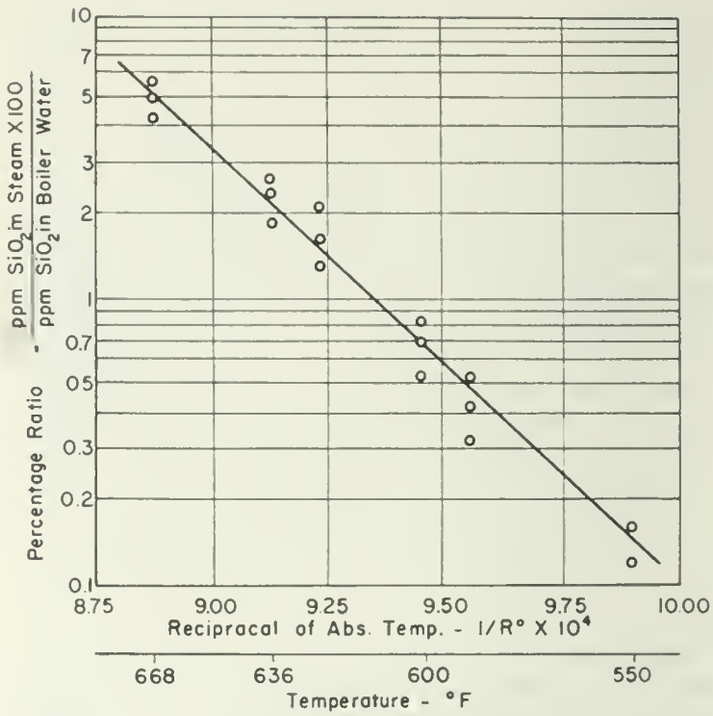


FIG. 65. —PLOT OF LOG OF SiO_2 RATIO VERSUS RECIPROCAL OF ABSOLUTE TEMPERATURE.

value of the boiler water is decreased.

When superheated steam is passed over solid silicic acid at various pressures and temperatures, definite amounts of silica are present in the steam corresponding to the temperature and pressure. No silica occurs in the superheated steam when it is passed over solid sodium silicate.

These results indicate that solid silicic acid has an appreciable vapor pressure at temperatures above 400 deg. F. When the silica is present in solution in the boiler water as sodium silicate, the mole concentration of the silicic acid is very low; consequently, the vapor pressure over the solution does not become appreciable until pressures around 1200 psi. are reached. The vapor pressure of the silicic acid over the boiler water will be low due to the small mole concentration and the high pH value of the boiler water. Thus, the concentration in the steam will be much lower than that corresponding to the true vapor pressure of the solid silicic acid at the higher temperature and pressure in the high pressure end of the turbine. However, as the temperature and pressure of the steam drop as it passes through the turbine, a point will be reached where the silicic acid concentration in the steam becomes greater than the silicic acid content corresponding to the vapor pressure over solid silicic acid. When this occurs, silicic acid will deposit. Thus, the higher the silica content of the steam leaving the boiler, the higher up in the turbine the deposits will start forming, and the greater the amount of deposit which will form in the turbine.

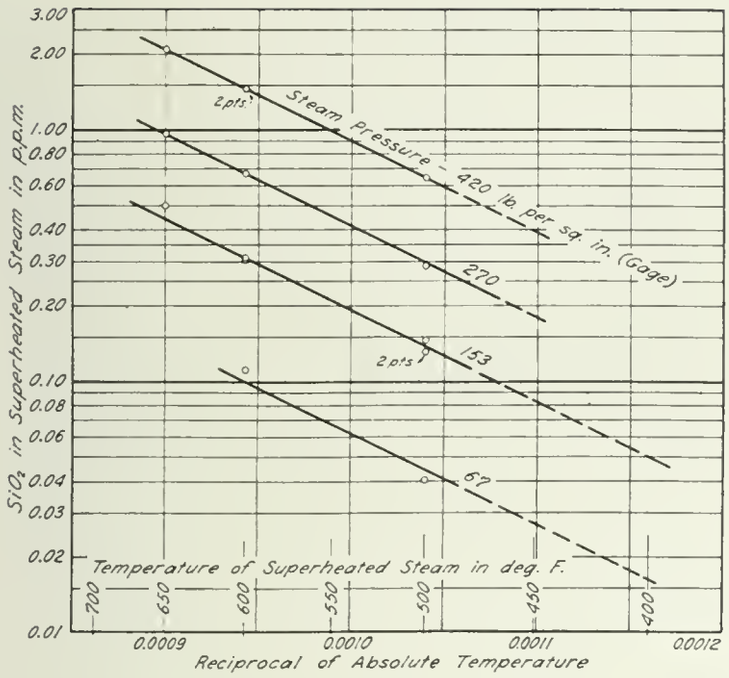


FIG. 66.—CONCENTRATION OF SILICA IN SUPERHEATED STEAM VERSUS RECIPROCAL OF ABSOLUTE TEMPERATURE.

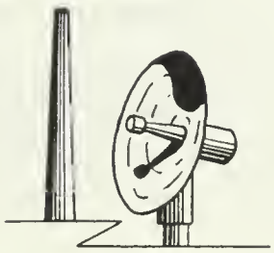
If the silica in the boiler water is kept below certain values, the deposit forming in the turbines should be negligible.

Tests conducted in several power plants gave ratios of silica in the steam to that in the boiler water almost identical with the ratios determined in the laboratory.

From the study made in the power plants, it was concluded that when the silica in the steam became higher than 0.1 ppm., there was a probability that deposits would form in the turbines. When this value has been kept below 0.05 ppm., the units have operated for over two years with no indications of deposits forming. This indicates that, in order to prevent this type of deposit, it is essential that the silica in the steam be kept very low. This may be accomplished by keeping the silica in the boiler water very low (below 5 ppm. at 1200 psi. or 2 ppm. at 2000 psi.). This in turn necessitates very good treatment of all make-up water to the boiler so as to reduce the silica entering the cycle to a very low value.

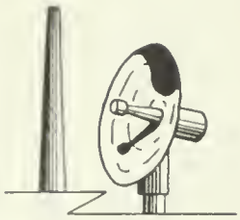
These few examples show how the principles of physics and chemistry have been applied in solving problems in the boiler feed water conditioning.

RADAR — WEATHER



Program Chairman

GLEN E. STOUT



Radar-Weather

All sessions will be held at Allerton Estate.

MONDAY, OCTOBER 1

Afternoon Session
A. C. Bemis, Presiding

2:00 "Census of Radar Weather Projects," A. C. Bemis
3:20 "NRL's R5D," I. Katz
3:30 "The Joint Commission on Radio Meteorology," J. S. Marshall
3:45 "Determination of Cloud Bases and Tops by Radar," W. Gould
4:15 Discussion

* * *

Evening Session
H. R. Byers, Presiding

7:15 "Details of the Production Model AN/CPS-9 Radar," W. J. Schiff and E. L. Williams, Jr.
7:30 "Comparison of Average Radar Signal Intensity with Rainfall Rate," P. M. Austin
8:00 "Quantitative Radar Rainfall Problems," G. W. Farnsworth
8:30 "Measurement of Point and Areal Rainfall by Radar," D. Jones and H. Hiser
9:00 Discussion

9:30 Informal get-together

Radar-Weather

TUESDAY, OCTOBER 2

Morning Session
W. L. Everitt, Presiding

8:30 "Reduction of Fluctuations in Echoes from Randomly Distributed Scatterers," J. S. Marshall and W. Hitschfeld

9:00 "The Present Uses and Limitations of Rasaph," A. Fleisher

9:30 "Microwave Scattering from Nonspherical Hydrometeors," D. Atlas

10:00 "Effects of Attenuation on Range Performance of Radar Set AN/CPS-9," D. Swingle

10:30 Discussion

Afternoon Session
J. S. Marshall, Presiding

1:30 "Theory of Radar Upper Band," R. Wexler

2:00 "Airborne Raindrop Size Measurement and Instrumental Techniques," R. M. Cunningham

2:30 "Results of Raindrop-Size Measurements," R. Boucher

3:00 Discussion

3:30 Inspection of M. I. T.'s B-17 at University of Illinois Airport

* * *

5:30 Reception, Illini Union Building

6:30 Banquet, Illini Union Building

Radar-Weather

WEDNESDAY, OCTOBER 3

Morning Session

Panel Discussions: 8:30 a.m. - 11:30 a.m.

1. Topic: New Developments in Using Radar for Hurricane Tracking

   Moderator: R. C. Jorgensen

   Panel Members: G. Dunn
   M. Latour
   J. Anderson

2. Topic: Suggested Fields of Study

   Moderator: A. C. Bemis

   Open Discussion

CENSUS OF RADAR-WEATHER PROJECTS

BY A. C. BEMIS*

The summary presented here was compiled from the remarks made by Mr. Bemis at the conference and the replies to a questionnaire sent out by Mr. G. E. Stout.**

WEATHER-RADAR RESEARCH PROJECTS

Microwave Propagation

1. Central Radio Propagation Laboratory, National Bureau of Standards (H. Bussey).
2. Cruft Laboratory, Harvard University, Office of Naval Research (H. R. Mimno).
3. Electrical Engineering Research Laboratories, University of Texas, Office of Naval Research (J. R. Gerhardt).
4. Naval Electronics Laboratory, San Diego (L. J. Anderson).
5. Wave Propagation Research Branch, Naval Research Laboratory, Anacostia (M. Katzin).

General Weather Radar

6. Geophysics Research Division Air Force, Cambridge Research Center (D. Atlas).
7. MacDonald Physics Laboratory, McGill University, U. S. Air Force and DRB, CAN (J. S. Marshall).
8. Meteorological Branch, Signal Corps Engineering Laboratories (W. B. Gould).
9. Weather Radar Research, Massachusetts Institute of Technology, Signal Corps Engineering Laboratories (A. C. Bemis).

Cloud Physics, Hydrology, and Storm Detection

10. Cavendish Laboratory, Cambridge University, England (J. E. N. Hooper).
11. Division of Radio Physics, Commonwealth Scientific and Industrial Research Organization, Chippendale, Australia (E. G. Bowen).
12. Cornell Aeronautical Laboratory, Office of Naval Research (S. Chapman).
13. Engineering and Industrial Experiment Station, University of Florida (M. H. Latour).
14. Mt. Washington Observatory, Harvard University, Air Force (R. Wexler).
15. Physics Department, New Mexico School of Mines (E. J. Workman).
16. Project Cirrus, General Electric Research Laboratory (I. Langmuir).
17. State Water Survey, University of Illinois (G. E. Stout).
18. Woods Hole Oceanographic Institution (A. H. Woodcock).

WEATHER-RADAR DEVELOPMENT PROJECTS

Instrumentation

19. Airborne Instruments Laboratories, Office of Naval Research (R. W. Miller).
20. Air Weather Service, Air Force (G. A. Guy).
21. American Airlines and Air Transport Association, Office of Naval Research and National Advisory Committee for Aeronautics (F. C. White and D. B. Talmage).
22. Cook Research Laboratories, Air Materiel Command (J. C. Bellamy).
23. General Electric Company, Air Materiel Command (D. S. Davidson).
24. Glenn L. Martin Company, Air Force (E. F. Hill).
25. Raytheon Manufacturing Company, Signal Corps Engineering Laboratories (E. L. Williams).
26. Weather Radar Branch, Signal Corps Engineering Laboratories (W. J. Schiff).
27. General Electric Company, Electro-Mechanical Division.

Application

28. Air Weather Service, 16th Weather Squadron, Chanute Air Force Base (H. W. Silk).
29. Dow Chemical Company, Freeport, Texas (R. C. Jorgensen).
30. A. H. Glenn and Associates, New Orleans, La. (A. H. Glenn).
31. Public Service Co. of Northern Illinois,

*Project Supervisor, Weather-Radar Project, Massachusetts Institute of Technology, Cambridge, Mass.
**Meteorologist, Illinois State Water Survey, Urbana, Illinois.

and Commonwealth Edison Co. (J. L. Wysong, H. L. Garton).

32. U. S. Weather Bureau.

* * *

1. Central Radio Propagation Laboratory, National Bureau of Standards
   H. Bussey

2. Cruft Laboratory, Harvard University
   H. R. Mimno

The Office of Naval Research is sponsoring mathematical investigations of reflectivity and propagation at microwave frequencies. Some laboratory work is conducted.

3. Organization: University of Texas, Electrical Engineering Research Laboratory, Austin, Texas
   Name: The Propagation Characteristics of Centimeter and Millimeter Radio Waves
   Director: A. W. Straiton
   Sup. Agency: Office of Naval Research, Contract Nonr. 375 (01), formerly N5ori-136 PO1
   Purpose: To study the transmission characteristics of centimeter and millimeter waves through the lower atmosphere and to determine, through measurements of the angle-of-arrival of radio energy and height gain and distance profiles, the degree to which refraction, scattering, and reflection from elevated layers are effective in producing the observed signal strength and phase distribution.

4. Naval Electronics Laboratory, San Diego, California
   L. J. Anderson

5. Organization: Naval Research Laboratory, Navy Dept., Washington 20, D. C.
   Name: Wave Propagation Research Branch
   Director: Martin Katzin
   Purpose: Fundamental research in radio wave propagation as related to radar

General Description: This group is interested in the over-all propagation problem. It investigates such phenomena as sea return, target properties, radar weather, and radar refraction. The primary investigating tool is a flying laboratory, an R5D, equipped with four radars on different wave lengths from 3 through 20 cm. These are especially designed radars which can record pulse to pulse returns from targets. The R5D will have a 15-foot vertical retractable meteorological mast which contains instrumentation for measuring drop size and distribution in rain, total liquid water in rain, dry bulb-, wet bulb-, and dew point temperatures. It is anticipated that the installation of this radar and the meteorological equipment will be completed in 1952, when collection of data will begin.

6. Geophysics Research Division, Air Force Cambridge Research Center
   Director: David Atlas

Work in the Tropospheric Branch of the Atmospheric Ionization Laboratory of the Geophysics Research Division has been directed toward the use of electromagnetic radiations in studying the lower atmosphere with particular emphasis in the microwave region and its use in the observation and study of clouds and precipitation.

I. Interpretative Studies: aimed at learning the nature of microwave reflections from various types of atmospheric hydrometeors.

   A. The interpretation of microwave reflections from rainfall.
   B. The microwave determination of particle size distributions in clouds and precipitation.
   C. The statistical nature of microwave reflections from clouds and precipitation.
   D. Microwave scattering from nonspherical hydrometeors.

II. Cloud Physics Studies: directed at the interpretation of microwave reflections in terms of the physical characteristics and processes within clouds and precipitation.

   A. 1 cm. microwave instrumentation makes it possible to detect some clouds in the early stages of development.
   B. Instruments for use with radar are pulse integrators, pulse fluctuation analyzers, facsimile recorder, cameras, etc.
   C. Some special instruments are Zenith scanning antenna system, instruments for the determination of polarization characteristics, and accurate signal measuring equipment.

7. Organization: Stormy Weather Research Group, McGill University, Montreal, Canada
   Director: Dr. J. S. Marshall, Associate Professor of Physics
   Sup. Agency: Defence Research Board of Canada (Grant No. 99) which provides equipment and maintenance only.
   Purpose: Study of precipitation and cloud by radar and other physical means

General Description: An AN/TPS-10A radar is operated at the Montreal Airport, Dorval, which, incidentally, renders a happy liaison with the Meteorological Service of Canada possible. The radar is used for quantitative analysis of precipitation, leading to knowledge of structure and formation.

At present, radar studies include investigations of depolarization and fluctuations in the radar return, and of smoothing procedures. The radar observations suggest allied experiments in the laboratory and in the field. A new method of finding cloud heights by stereoscopic photographs has recently been developed. In the laboratory, experiments on the microphysical processes of precipitation are carried out; some of these are the studies of coalescence, evaporation and heat transfer among rain, snow and cloud particles. Velocities of fall and the size distribution of snow particles are being investigated.

The present project at McGill is a continuation of the original "Stormy Weather" research project, which began in Ottawa in 1945 as part of the Canadian Army Operational Research Group. Results have been reported in eight CAORG reports, five M. S. and three Ph. D. theses.

8. Meteorological Branch, Signal Corps Engineering Laboratories
   W. B. Gould

Sponsors several contracts with universities in determining the importance and application of radar in meteorology. Assisted with the development of the CPS-9.

9. Organization: Massachusetts Institute of Technology, Cambridge, Massachusetts
   Director: A. C. Bemis
   Sup. Agency: Signal Corps Engineering Laboratories, Belmar, New Jersey, Contract No. DA-36-039-sc-124
   Purpose: To explore the applications of radar in meteorology, to improve weather radar techniques, and to learn to interpret more completely all types of weather information which may be contributed by radar systems.

General Description: The project operates four radar systems: AN/CPS-9 (XE-2), SCR-615-B, AN/TPS-10A, SCR-584.

The CPS-9 and SCR-615 are carefully calibrated and equipped with Pulse Integrators for measuring weather echo intensities. In addition, they can be equipped with a Box Car Generator, Rasaph, individual pulse recording equipment, and other devices for studying in accurate detail the fine structure of weather echoes. Autocorrelation and spectrum analysis techniques are both used.

All radars are equipped with automatic cameras.

B-17 No. 9281 is a thoroughly instrumented aircraft with both standard and research meteorological instruments, and has been making weather flights in conjunction with radar observations for five years.

A special, high-sensitivity rain gage, rain drop size equipment, and radar winds aloft gear are operated 12 miles from our main radar site.

At present we are concentrating on (I) a study of the basic nature of weather echoes, their average intensity and pulse-to-pulse variations; (2) the relation between the structure of weather echoes as presented on scopes, and the storm structure causing them.

10. Cavendish Laboratory, Cambridge University, England
    J. E. N. Hooper

Radar is used as a tool in cloud physics studies.

11. Division of Radio Physics, Commission of Scientific and Industrial Research, Australia

Radar is used to study precipitation patterns, "bright band" effects, etc., and the observations integrated into cloud physics studies.

12. Organization: Cornell Aeronautical Laboratory, Inc., 4455 Genesee St., Buffalo, N. Y.
    Name: Thundercloud Electrification Studies
    Sup. Agency: Office of Naval Research, N6ori-119, Task Order 13, Geophysics Branch, Code No. NR082056
    Purpose: Four prime phases of this project are: electric field measurements, snow storms, snow electrification, and rain drop electrification.
    Equipment: An SG-6 (10 cm. surface search radar) was used to spot snow flurries and get ready for radiosonde releases. Used radiosonde equipment, rain drop tube, specially modified radiosonde equipment, snow meter, DC amplifiers, etc. The final report is now in progress.

13. Organization: Engineering and Industrial Experiment Station, University of Florida
    Director: M. H. Latour
    Supported by: State of Florida funds. Equipment on Bailment Contract from the Air Materiel Command.
    Purpose: A. Investigation of rainfall intensity
    B. Investigation of microwave attenuation
    C. Study of subtropical weather
    Facilities: The radar equipment available consists of one S-band radar, the SCR 615B, two X-Band units, the AN/APS-15 and the AN/APQ-13. The fixed radar unit has just been relocated on the campus of the Uni-

versity on a 120-foot tower. New buildings have been erected to house the equipment and to serve as a laboratory.

Radar studies have been made of four tropical cyclones. Several radar observations of squalls and frontal conditions have been made, including one of a tornado.

The rainfall and attenuation studies will be resumed now that the equipment is located at the improved site.

14. Mt. Washington Observatory, Harvard University

Cloud physics studies are being conducted under Air Force sponsorship. Rain drop size measurements and their relationship to rainfall intensities and synoptic situation studied.

15. New Mexico School of Mines, Physics Dept.
E. J. Workman

Cloud physics studies are being conducted. Electrical potentials developed in freezing solutions, effective life of silver iodide particles, reactivation of silver iodide particles, and ammonia as an agent for reducing electrical potentials in clouds are a few phases of the work. Radar is used for observing rainfall patterns and evaluating the effect of agents introduced into clouds.

16. Project Cirrus, General Electric Research Laboratory
I. Langmuir

This extensive project is essentially a study of precipitation mechanisms and the effects of various "cloud seeding" techniques.

17. Organization: Illinois State Water Survey, Urbana, Illinois
Name: Quantitative Rainfall-Radar Study
Director: Glenn E. Stout
Purpose: To determine the utility of radar for water resources applications, to develop equipment and technique for determining areal rainfall with radar. One APS-15 is installed at El Paso, Illinois; a second APS-15 set and a TPL-1 are located at the University of Illinois Airport. Each set has a scope camera and an automatic system to control the receiver sensitivity for determining isoecho patterns. 34 Friez 12.648" weighing bucket rain gages are used to check the surface rainfall patterns.

18. Organization: Woods Hole Oceanographic Institution, Woods Hole, Massachusetts
Name: Atmospheric Sea-Salt Studies
Director: Alfred H. Woodcock
Sup. Agency: Office of Naval Research, NR-085-001
Purpose: To obtain the size distribution and number of atmospheric sea-salt particles near the sea surface, in the sub-cloud and cloud layers and determine its importance in producing heavy rainfall. Simultaneous measurements of rain drop size distribution and salt concentrations will be made both from the ground and from aircraft at and below the cloud level.

19. Airborne Instrument Laboratories, Office of Naval Research
R. W. Miller

20. Air Weather Service
G. A. Guy

Responsible for locating, organizing, and maintenance and training program for the CPS-9 radar.

21. Organization: National Advisory Committee for Aeronautics, 1724 F. Street NW, Washington 25, D. C.
Director: Donald B. Talmage
Purpose: Currently engaged in instigating projects to evaluate the use of Airborne Radar in thunderstorms and line squalls. Up to present, we have cooperated with American Airlines and the Navy in preliminary projects and have made tentative plans to cooperate in a joint project with the Naval Air Test Center to continue the work.

22. Organization: Cook Research Laboratories, 1457 Diversey Parkway, Chicago 14, Illinois
Name: a. Lightning Avoidance
b. Weather Reconnaissance Systems Study
Director: Dr. J. Robert Downing
Associate: Dr. John C. Bellamy
Sup. Agency: Air Materiel Command
(a) Contract No. AF33(038)-23715
(b) Contract No. AF33(038)-23324
Purpose: (a) Study of means by which aircraft in flight may detect and avoid lightning areas.
(b) Suggested by title.

23. General Electric Company, Air Materiel Command
D. S. Davidson

24. Organization: The Glenn L. Martin Co., Baltimore, Maryland
Name: Airborne Weather Detection Radar
Director: E. F. Hill, Jr., Project Engineer
Sup. Agency: U.S.A.F., Contract No. AF33

(038)-19562, classified Confidential
Purpose: Development of a radar system

25. Raytheon Manufacturing Company
Project Engineer: E. L. Williams

Production of the CPS-9, a 3 cm. high-power radar developed by the Signal Corps as a storm detection radar.

26. Weather Radar Branch, Signal Corps, Engineering Laboratories
W. J. Schiff

Responsible for the design and development of the CPS-9 radar.

27. Organization: General Electric Co., Electro-Mechanical Division, Schenectady, N. Y.
Name: Development of "Airborne Cloud Base and Top Indicator - AN/APQ-39"
Director: S. M. Kaplan
Sup. Agency: Air Force, Contract No. AF33 (038)-14299
Purpose: To permit the continuous recording of the location and density of cloud layers above and below a surveying aircraft.

28. Organization: Detachment 16-IL, 16th Weather Squadron, Chanute Air Force Base, Illinois
Name: Storm Detection
Director: Capt. H. W. Silk, Jr., Detachment Commander
Purpose: To furnish additional weather data for dissemination to agencies of the weather network; and to enable our local forecasters to maintain a more detailed, current, and dynamic picture of hydrometeors associated with storms.
Equipment: AN/APQ-13 Airborne Type Search Radar modified for storm detection.

29. Organization: Dow Chemical Company, Electrochemical Engineering Division, Freeport, Texas
Name: Industrial Radar for Hurricane Tracking
Director: Oliver Osborn
Purpose: To help the Weather Bureau provide Gulf Coast industry with better hurricane warnings. Existence of the radar has improved employee relations by increasing confidence in adequate storm warnings and by prevention of unnecessary layoffs because of premature plant shutdown.
Equipment: SCR 784 and SCR 527

30. A. H. Glenn and Associates, New Orleans, Louisiana
A. H. Glenn

Using radar as an aid in forecasting the weather for offshore drilling operations.

31. Public Service Company of Northern Illinois, and Commonwealth Edison, Chicago, Ill.
J. L. Wysong, H. L. Garton

These companies are conducting a cooperative investigation. A 3 cm. radar (APS-15) is being set up at the Chicago Municipal Airport and will be used to evaluate the use of radar for sleet warning and dense cloud coverage warning.

32. U. S. Weather Bureau

The Weather Bureau is in the process of setting up an extensive radar system for storm warning and forecasting purposes. Several stations are already installed and many more are planned. Most of the equipment is 10 cm. (APS-2). Also radar reports are received from several individual cooperating organizations.

NAVAL RESEARCH LABORATORY'S R5D

BY ISADORE KATZ*

WITH DISCUSSIONS BY A. C. BEMIS, D. ATLAS, H. R. BYERS, D. B. TALMAGE, AND I. KATZ

The Wave Propagation Research Branch of Naval Research Laboratory is interested in basic and applied research in microwave propagation. Among the many problems involved under this subject is the problem of electromagnetic attenuation and scattering by hydrometeors in the atmosphere. For the purpose of carrying out such studies we feel that one fruitful approach is to combine the radio and meteorological equipment in one airplane and make almost simultaneous measurements of the electromagnetic signals and weather phenomena. To fulfill this plan we are now modifying and equipping an R5D (commercial equivalent: DC-4, Air Force designation: C-54) in the manner I shall now describe.

We carry four radars on wave lengths between 3 and 24 centimeters, which have a high degree of versatility, along with two basic meteorological tools, a disdrometer and a total water collector. In addition, we are installing auxiliary equipment which should provide information of interest to the meteorologist.

Fig. 67 is a photograph of a model of our modified plane as it appeared in recent wind tunnel tests. The plane is 94 feet long and has a wingspread of 118 feet. Its speed range will be approximately 100 to 150 mph. You may note the 4 nacelles attached under the wings. These nacelles are 56 inches across and about 12 feet long. Each nacelle houses a 4-foot paraboloid, the same size for each of the four synchronized radars. This results in beam widths of about 2 degrees for X-band, 6 degrees for S-band and 15 degrees for L-band. At present we are planning to install two X-band radars differing in frequency by only 30 mc. These dishes are mounted on pedestals which, with proper servo control, provide 3-axis stabilization. Therefore, within certain limits, regardless of the plane's motion, all four antennas will be looking at a predetermined area. The stabilization controls have a sensitivity of about 1/4 degree. We have a scanning rate of 30 rpm., maximum. Each radar is designed to transmit and receive any polarization: horizontal, vertical, circular** and elliptical. This is achieved by the use of two receivers, each being fed one of the two orthogonally polarized components of the returning signal. The output of each receiver is fed ultimately into one gun of a two-gun oscilloscope; both components then are recorded side by side.

Fig. 68 is a view of the model showing a close-up of the 15-foot meteorological mast. The mast is retractable to enable the operator to service the equipment while in flight. In addition, it is planned to fly with the mast retracted whenever flights are being made for which no meteorological measurements are needed. Streamlining of flow is effected by means of the fairing as shown. When the mast is in this retracted position, the operator may open a door in the fuselage and gain entry into the mast enclosure. This gives him access to some of the equipment and to the junction boxes for all the instruments; this enables him to make continuity tests, tests for shorts, etc.

Fig. 69 shows the mast in its flying position. The mast dimensions are 24 inches long and 6 inches at its widest point. The uppermost instrument is the disdrometer. It is shielded from any turbulent flow which may exist around the mast by a smoothing plate which extends five inches forward of the mast structure. Capillary collectors are mounted on wing-like structures below and to the side of the

FIG. 67. —MODEL OF MODIFIED R5D.

*Physicist, Naval Research Laboratory, Washington, D. C.

**Chait, H. N., Microwave Radar Antenna, Electronics, March 1951.

FIG. 68. —CLOSE-UP OF RETRACTABLE METEOROLOGICAL MAST.

disdrometer. All of you are familiar with the porex-type collectors which we use to sample the total liquid water in the rain. Connected directly to and situated about six inches below the collector itself is a mercury filament flow meter. This will be described in some detail later. Below the capillary collector you will see the Venturi tubes which are used to provide the vacuum needed in operating the disdrometer. And now, in order as we look down the mast, we find intakes for a wet-bulb thermometer, a dry-bulb thermometer (together these constitute our design of the psychrograph), a vortex thermometer (designed at NRL with a spiral nose-piece), a refractometer (when one is perfected for flying), and a dew-point hygrometer. Two important instruments which are not mounted on the mast but which I might mention at this point are electric field meters (mounted one above the fuselage and one below, at a distance of about 30 feet forward of the mast) and accelerometers mounted inside the plane at the center of gravity.

Fig. 71 is a sketch of the starboard side of the radar-weather control compartment. In it are shown the four radar operator positions, four "Control" racks, and a work bench for making emergency repairs during flight. In the control racks are such equipment as power supplies, timing circuits, synchronizing circuits, stabilizing controls, etc.

Fig. 70 is a sketch of the layout for the port side of the same compartment. Looking first at the forward portion you see the recording equipment. For the radars there are 12 slide-out shelves which hold the dual-gun oscilloscopes and cameras. PPI information and multi-pulse data are recorded on Fairchild Type A cameras while single pulse A-scope data are recorded with Fairchild-Dumont Oscillograph Record cameras. We have recorded, then, single pulses on all four radars, and in addition, any desired number of pulses superimposed on a single exposure.

The fifth column contains recording equipment for the meteorological instruments. The top sliding shelf contains a dual-gun oscilloscope on which will appear pulses from the disdrometer. On the second shelf is a Brown 12-point temperature recorder. Its primary purpose is to record dew point but it can be used for any other temperature desired. On the lowest shelf are two Esterline-Angus recording meters. One is a twenty-pen recorder, which is an integral part of our flow meter; the other is a 0-1 milliampere meter which records the output of the psychrograph. In the next column in this series of racks are two data panels which are photographed once every second. These data panels contain dials, meters, lights, clocks and counters; on these panels are recorded such information as range, pulse length, repetition rate, scanning speed, error lights, time, elapsed time, altitude, air speed, etc.

In the next position you will see what we have termed the "Master Operator." From this position the combined radar weather flight operation is controlled. The master operator has four PPI scopes in front of him so that he can see what is happening on any one or all four radars. He has three separate intercommunication systems available so that he may speak to the pilot, to the radar operators, or to the meteorologists. He controls

FIG. 69. —METEOROLOGICAL MAST IN FLYING POSITION.

FIG. 70. —PORT SIDE OF RADAR-WEATHER CONTROL COMPARTMENT.

FIG. 71. —STARBOARD SIDE OF RADAR-WEATHER CONTROL COMPARTMENT.

the power to the various operations; he sets azimuth, elevation and range of the radars; he selects the proper operational technique for any particular run and synchronizes the various operations: those of the pilot, the radars and the meteorologist. He has error lights to indicate, for instance, when the antennas are given a command then cannot follow; when he sees an error light flash on, he takes immediate steps to correct the discrepancy. He also has "Ready" lights to indicate when preparatory operations are complete.

Immediately behind the master operator is the "Met" operating position. From this point all the meteorological instruments are controlled. The instruments are turned on, monitored and regulated from controls on these panels.

Fig. 72 is an enlarged sketch of the meteorological operator's position. Just above his writing table you may note the control panel, which contains switches, meters, dials and lights so that he may monitor all the instruments. Above this are shown the electric field meters and the vortex thermometer. To the left of the control panel are the nitrogen pressure valve, mast controls and oscilloscopes. One scope is a disdrometer monitor, and the other is a scope on which can be put PPI information from any of the radars. The latter scope provides the meteorologist with information which will enable him to determine in which area measurements are to be made. In the rack behind him are the power supplies, and all the electronics for the disdrometer and the psychrograph. On the shelf to his left is the dew-point hygrometer. Above and to the right of the hygrometer is his observation window.

I shall not go into a detailed discussion of the meteorological instrumentation at this time. However, it might be well to mention a little more about the disdrometer and the total water collector.

Our disdrometer is similar in most features to the original M. I. T. disdrometer. For laboratory tests we need an electronic calibrator and counting circuits in addition to an oscilloscope and recording camera. For calibration an air gun was mounted to fire accurately measured glass beads and metal bearings through the light beam of the recording head. Our last flights with this instrument gave results which indicate that the disdrometer is still reading unreasonably high as compared with the more acceptable values obtained by the capillary collector.

FIG. 72. —METEOROLOGICAL OPERATOR'S POSITION.

Fig. 73 is a schematic diagram of the flow meter we are building to measure small rates of flow as obtained by the capillary collector. Water enters the system through the porex collector, goes through a 4-way stopcock into the metering section, leaves through the stopcock and is caught in a reservoir. As water starts to flow through the metering section, it pushes a filament of mercury along with it. A light source and tiny phototubes (1N77) are so arranged around the glass tubing that as the mercury moves in front of a phototube its light is cut off. This information is sent by means of electronic switches to the multi-pen recorder and also the "Met" panel. When the mercury reaches the last phototube a solenoid is activated which reverses direction of flow in the metering section; the flow in the system continues uninterruptedly until the mercury reaches the last phototube on the opposite side. By means of a second solenoid, flow direction is again reversed and the cycle begins to repeat. It is planned to have the measuring and pressure compensation section of this system mounted in the mast.

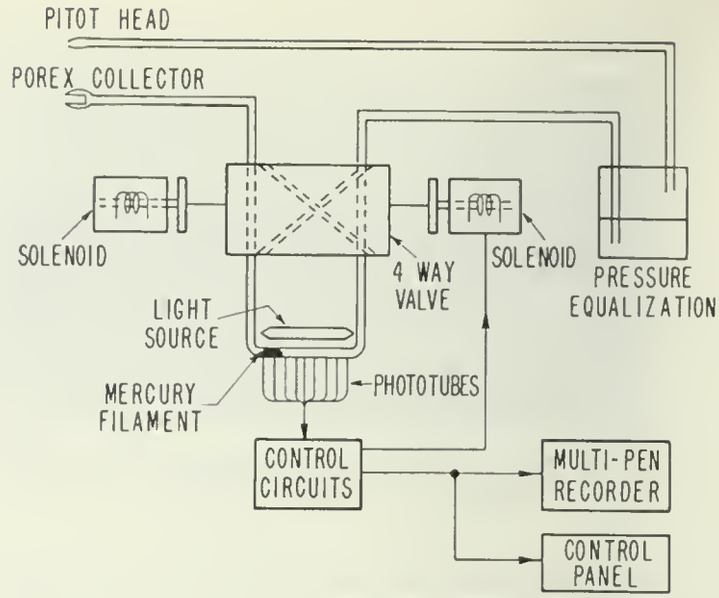


FIG. 73. —SCHEMATIC OF FLOW METER.

We hope that when data collection begins with this R5D we will be able to contribute substantially to existing theory of the scattering and attenuation of microwaves in the atmosphere; in addition, we hope to shed more light on how to use radar more effectively in meteorology.

DISCUSSION

A. C. BEMIS. —That was certainly a very impressive and extensive job of instrumentation on aircraft. As Katz pointed out, we are particularly interested in the weather instruments upon which we have been working quite actively with the Signal Corps at the M. I. T. project, over pretty much the same period and have come up with similar answers. Certainly, if we can get these gadgets to the stage where they can be made more readily available, it would be a tremendous boon to this general field of science in measurement because liquid water content is something which should be measured on a routine basis, not just by us research people. I would like to have some comments.

I. KATZ. —In the future we may consider replacing the second X-band radar with a 6-centimeter radar if components become available.

D. ATLAS. —How do you plan to analyze these data?

I. KATZ. —When the information is obtained in the form of amplitude versus time, we are in a position to use semi-automatic computers for analysis. These machines are extremely useful for the data obtained on droplet sizes by the disdrometer, for instance. For the larger bulk of data obtained by the radars, however, these machines cannot be used. We are acutely aware of the magnitude of the analysis problem and are actively working on the design of equipment for automatic analysis of the radar data.

H. R. BYERS. —(Question on exposure of instruments.)

I. KATZ. —Before deciding on the exposure of our instruments we conferred with aeronautical engineers of the Bureau of Aeronautics to determine the most feasible placement of instruments from the known flow lines around the R5D and other considerations. Ten feet from the skin of the airplane was decided upon as a safe distance. In addition, since the flow lines from the engines extend downward, it was decided to expose the instruments above the plane. The final solution was a single retractable mast, 15 feet high, to house all the meteorological instruments except the electric field meters and the accelerometers.

Since a scale model of this instrument now exists, it would be a simple task to test experimentally the actual flow configuration about this airplane with the modifications as shown in the figures.

A. C. BEMIS. —One thing that people ought to consider, though, in flying through the rain, is that the conditions may be somewhat different due to splash and so forth. I do not know if anyone knows just what goes on under those conditions, how far the splash effects extend beyond the skin of the ship. I am not suggesting they go 15 feet above it, but it seems that one should be a little cautious about it.

D. B. TALMAGE.*—(Question on gross weight.)

I. KATZ. —The maximum allowable weight for this R5D is 72,000 pounds. With the equipment as planned at present we will exceed this figure with full gasoline load. Therefore, for long overwater flights en route to an experiment, when larger amounts of fuel are needed, we intend to carry less equipment. The equipment we remove will be shipped on other aircraft and reinstalled before the experiment.

D. B. TALMAGE. —When you are running through a thunderstorm, what about all the hail and extraneous material struck?

I. KATZ. —We feel that most of the information we need may be obtained without resorting to flights into thunderstorms. In the past our flights have been through small tropical showers. With such isolated targets we were more easily able to synchronize our radar and meteorological data. In large thunderstorms, with their well-known cellular structure, it would be more difficult to identify the exact target when analyzing the records.

*Research Scientist, National Advisory Committee for Aeronautics.

BRUSSELS MEETING OF THE JOINT COMMISSION ON RADIO METEOROLOGY

BY J. S. MARSHALL*

WITH DISCUSSIONS BY A. C. BEMIS, R. M. CUNNINGHAM, H. R. BYERS, W. B. GOULD, L. J. BATTAN, J. S. MARSHALL

There is an impressive array of International Scientific Unions. Radio has its U. R. S. I., while the Meteorological Association is a component of the U. G. G. I.: the International Union of Geodesy and Geophysics. By way of a superstratum, there is a Council of International Scientific Unions.

Now, as is the way with international organizations, these unions tend to establish committees and commissions. (There was a meeting at Brussels this summer, for instance--a committee on the age of the earth. It may well be that the matter is felt to have achieved some urgency now, perhaps toward the preparation of a suitable obituary. But that is by way of example.) The group most relevant to us assembled here is the Joint Commission on Radio Meteorology. The ten members of this Commission represent the Radio Scientific Union, the Union of Geodesy and Geophysics, and the Union of Pure and Applied Physics. It seems to have been felt by these bodies that the boundary layer of research activity shared by microwave propagators, atmospheric electricians and physical meteorologists should be reviewed on a world-wide basis. What happened was that, in the course of two days, seven good invited papers in these various fields were presented to a select group of less than twenty people.

There is no point in my discussing the two papers in Radar Weather, for their subject matter will be covered more fully at the present meeting. It may be of interest to note some relationships, however, between our immediate field and other fields of endeavor discussed before the Commission and before the U. G. G. I.

Lloyd Anderson of the Navy Laboratory, San Diego, reviewed microwave attenuation by precipitation and by atmospheric gases. This, of course, is right next door to our own business. Most of the attenuation is by rain, and it is the same rain that provides our weather echoes. The same size distributions are relevant, the same patterns of rainfall. The electromagnetic theory involved is, of course, closely related. Indeed, there is the same factor two discrepancy, or is it two at the moment, between observation and theory. Attenuation studies are something that those of us who are here should watch closely, something in which we might well participate.

Point-to-point attenuation studies lack one bright facet in which we students of scattering can revel. That facet is noise. If they transmit a pulse, they receive a pulse, a poor attenuated one, but still a pulse. When we transmit a sharp, square pulse, we receive a continuum of pure noise. The amplitude and phase of our received signal are randomly varying quantities.

Now, of course, the propagationists have their randomly varying quantities too. Professor Sheppard of Imperial College discussed turbulence in the low atmosphere, while Gordon of Cornell spoke about inhomogeneities in temperature and humidity. Both of these are random quantities. Randomness is a very real thing, the same sort of thing whether it exists in the movements of the atmosphere, in the positioning of raindrops, or in the rattling around of electrons in a resistor. Any new advance in analyzing randomness, in understanding randomness, is likely to be applicable in many fields. We, with our own stochastic speciality, should bear this in mind.

The other two papers presented were about lightning. (The same lightning, this may be, that Ligda sees by radar dancing on the top of his showers.) Lugeon of Switzerland talked about SFERICS, the technique of storm location by direction-finding on the long-wave radiation from lightning. His detection ranges are much greater than ours; his precision is very much less. It seems to me that a simple, relatively short range form of sferics would prove useful in conjunction with weather radar. Workman has, of course, made excellent use of something of the sort, presumably at very short range. Should we be using this type of thing more generally?

Norinder of Upsala talked about lightning too, about the wave form of lightning radiation at short and moderate ranges. After thirty years in this special field, Dr. Norinder approaches his prob-

*Associate Professor, Physics Department, McGill University, Montreal, Ontario.

lems with a vigor and freshness that is truly impressive.

There is a notable gap between Norinder's field and ours. He studies discharge. He seems to neglect, almost to avoid, the processes of charge generation and concentration that must precede his flash. That process is one aspect of the precipitation machine that we watch with our radars. We watch it, but we have a long way to go before we explain it, either as a watering can or as a Wimshurst machine.

Between Norinder and us there is an awful lot of cloud physics. There was not much cloud physics at our Commission meeting. There was enough to make up the next week, when the U. G. G. I. included a cloud physics session, with Dr. Bergeron as chairman. The most impressive equipment described was Dr. Ross Gunn's cloud chamber, 200 meters deep, built into an old mine shaft. The most exciting topic was the production of rain by coalescence. Dr. Bowen started the excitement in the opening paper, by saying that he had seen rain without ice crystals, seen it with his own eyes, on his own radar, in his own aircraft. The evidence was conclusive. The whole cloud was below the aircraft. The aircraft was below the freezing level. The clouds were convective, mind you. Stratus stuff inclines more toward Bright Band and Bergeron.

Bowen's evidence was conclusive. The explanation, coming in a following paper from Ludlam, of London, was on the whole convincing. Where did the rain come from? Not cold enough for ice, not time enough for condensation, it must have been coalescence. This much is surely true. But the problem of initiating the coalescence calls for some invention. Ludlam's invention began, if I remember aright, with largish drops of spray sucked up from an angry sea. These still were not big enough to grow by coalescence. (Not when they got sucked up.) But during their half-hour trip through the cloud they grew, until at the cloud top they were ripe for a fall, and for the rapid coalescent growth that is the lot of falling drops. But now invention begets invention. Having achieved big drops, but not enough of them, invention number two must be a multiplier, and the obvious choice is chain reaction.

The combined mechanism is a bit too complex, I feel, a bit too tight as to timing, to be convincing. It is a good theory, mind you. To the extent that they inspire doubt and experiment, these inventive theories are valuable. If they get placid acceptance, though, they may discourage experiment and observation. We cannot have that.

There seems to be good evidence for rain by coalescence, not only when the ice phase is absent, but also when it is present.

What do our radars say about it? The most notable phenomenon they reveal, to my mind, is the complete lack of any bright band, or any other freezing-level discontinuity, in many showers. Second, there is the sudden appearance of shower echoes of great vertical extent. I am not positive of this second observation, but I think it is the case. Now, this seems to be the open season for guessing, before the observers move in with the facts. In the light of these radar observations, and some notions about size distribution, I would like to make my own guess as to what sets off the coalescence of cloud to rain. My guess is turbulence. Could it be that as cumulus increases in height its turbulence, its microturbulence if you like, increases until a threshold is reached sufficient to dash the cloud drops together? If it could, then precipitation might suddenly appear over a wide range of heights. And does glaciation increase updraft, increase turbulence? If so, then the onset of glaciation, when it does occur, might be felt over a wide range of heights.

My suggestion comes as much from ignorance as from observation. It is simple; it does not involve sea-salt. It does reveal just how wide open the field is now. Let us play all the theories, and accept none, until observations give us the facts.

The next U. G. G. I., by the way is in Italy in three years' time, or so I understand. Dr. Burrow's Joint Commission will meet then too. In case we are still around, allow me to recommend these meetings to you.

DISCUSSION

A. C. BEMIS. —Thank you. That is a very inspiring and provocative discussion. It brings up many points that I would like to argue but I will wait my turn. The more we fly around in airplanes, the more I think we approach those lines of reasoning you have suggested for explaining the growth of rain drops. The person I thought I would call on for a comment here is Duncan Blanchard. I think he has been doing some work similar to Ludlam's on the sea salt theory and could come back with some good counter arguments. (Blanchard was not present.) Cunningham recently flew in a thunderstorm over Albany which seemed to fulfill your theories nicely. Isn't that right, Bob?

R. M. CUNNINGHAM.—A week ago, we (Weather Radar Research project's B-17) flew into a thundershower in which there was violent turbulence and frequent lightning. The tops of the cumulus congestus clouds were apparently only a few thousand feet above the freezing level. It seems to me that the coalescence process, helped by the violent small scale convection, caused most of the heavy rain reported by the ground stations.

H. R. BYERS. —I was interested in Dr. Marshall's comments about Dr. Norinder. It is true that he has not concerned himself with the dynamics of charge generation. There is too much speculation in that area, and therefore he has stayed out of it. Everyone who has worked in thunderstorm electricity has a theory on charge generation.

J. S. MARSHALL. —I wonder about the use of spherics with radar, if anyone here has done anything with it?

A. C. BEMIS. —Well, Herb Ligda has been doing some dreaming on that and talking with Gould about it. Gould is a weather-radar man in the Signal Corps spherics section so he ought to have some terrific ideas.

W. B. GOULD. —I have nothing much to say except that Mr. Ligda has told me about his letter which I never received. However, we have done considerable thinking about it. Don Swingle has attempted to do, on a limited scale, some correlation work between the spherics and the radar echoes, and the joint work that we have done with you at M. I. T. has been directed in that line. Herb Ligda has suggested to us the tying of a spherics indicator on the PPI so that we would have a flash along a line in the direction at which lightning occurred, thus connecting the lightning and rain very much as you suggested here this afternoon. I do not know why we did not think of it before, and I think we will try it.

L. J. BATTAN.*—At Cook we have been considering the construction of equipment combining radar with spherics. In view of present design considerations we want to use a receiver operating between 100 and 200 megacycles. We have had difficulty in finding spherics work carried on at this frequency. About 1939 Bell Labs. made some measurements. They used receivers operating at about 150 megacycles. If anyone here is familiar with any more work on atmospherics at these frequencies, we would be interested.

J. S. MARSHALL. —Do you use those frequencies to keep your range short?

L. J. BATTAN. —We were primarily thinking of getting suitable signals without having to get too large an antenna. The elimination of signals from thunderstorms at very large ranges also is desirable.

*Meteorologist, University of Chicago, Chicago, Illinois.

CLOUD DETECTION BY RADAR*

BY WILLIAM B. GOULD**

WITH DISCUSSIONS BY A. C. BEMIS, J. R. GERHARDT, D. ATLAS, D. D. REITER, M. L. STONE, R. C. JORGENSEN, S. E. REYNOLDS, D. M. SWINGLE, R. WEXLER, P. M. AUSTIN, M. G. H. LIGDA, W. B. GOULD

In the early part of 1945, personnel of Evans Signal Laboratory became interested in the possibility of detecting clouds by radar. Of course there was considerable radar storm detection background available at that time, but the storm detection equipments receive echoes from precipitation areas and do not consistently detect ordinary clouds. The two previous speakers have given information relative to ten-centimeter and three-centimeter storm detection sets. These equipments are not able to consistently detect ordinary clouds, and the probable reason for this failure is the small particle-size in clouds where the droplets are about 1/100 the size of rain drops. Theoretical investigations at Evans Signal Laboratory indicated that if a set operating on a wave length of approximately one centimeter had sufficient power and sufficient sensitivity, it should enable one to detect and locate clouds and cloud layers. These studies were based upon the concept that radio waves are scattered by cloud particles. There is another concept that considers the possibility that electromagnetic waves may be reflected by a surface discontinuity between a cloud and free air. It is believed that the cloud base and top indicator developed by the Signal Corps detects clouds chiefly by the scattering concept.

*Also presented at joint meeting of the American Geophysical Union and American Meteorological Society, April 20, 1949.

**Radio Engineer, Evans Signal Laboratory, Belmar, New Jersey.

An experimental radar set was constructed in the early part of 1945 and was operated on a wave length in the one-centimeter region. This equipment was sited so that its beam was directed vertically, and presentation was given on a type A-scope where altitude is denoted along a vertical line and signals from clouds are deflections to the right of this line. This equipment has detected clouds up to better than 45,000 feet. Numerous cloud layers have also been observed. It has been possible to locate clouds through several thousand feet of rain. As may be expected in very heavy rainfall, however, considerable attenuation takes place, and it may not always be possible to see the top of a storm. The minimum altitude that can be observed is about 800 feet. This limitation is due to the fact that the pulse length of the transmitted signal is of a definite length, and added to this fact is the recovery time of the receiver, which makes it impossible to read the bases of clouds at the lower altitudes. While A-scope indications give a very good picture of a cloud situation, it is impossible for an observer to remember such changes as may take place over a period of time. This fact brought up the need for some sort of recording device. After a survey of the field, it appeared that the more or less conventional facsimile recorder offered promise. Such a device was modified for a much slower paper speed (12 inches per hour). The recording gives one a plot of the altitude of cloud returns versus time which, in effect, is a vertical cross-section of cloud situations as they pass over a set.

FIG. 74. —AN A-SCOPE CLOUD BASE AND TOP PRESENTATION.

A sweep length of 40,000 feet with 5,000-foot markers is shown. The main pulse is at the bottom of the picture, occupying approximately 800 feet. The first cloud layer has a base of 11,000 feet, with a top of 12,500 feet. The higher deck gives a saturated signal with a base of about 14,000 feet, extending to 17,000 feet. Above this is a layer with a base at 20,500 feet, with the top measuring 23,000 feet. A thin layer exists at 25,000 feet. For accurate measurement of altitude, a movable range marker is used, enabling one to determine the altitude within a few hundred feet.

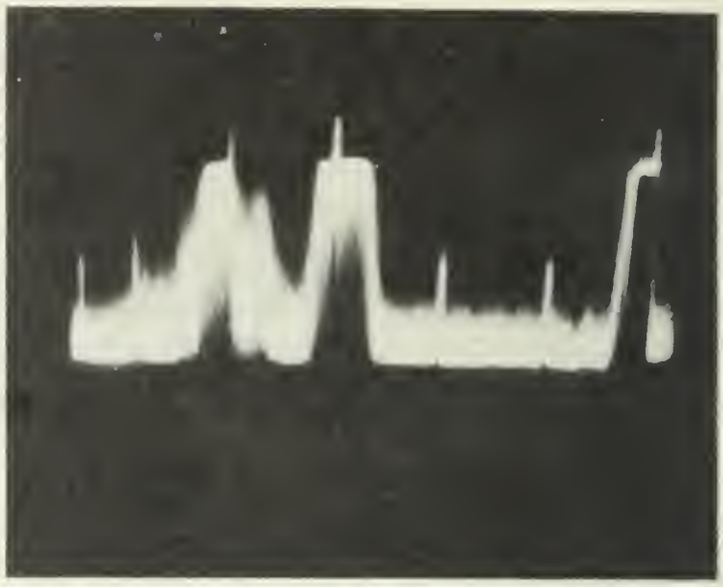


FIG. 75. —AN A-SCOPE CLOUD BASE AND TOP PRESENTATION.

A sweep length of 30,000 feet is depicted. The base of the bottom cloud deck is at 13,000 feet, with the top measuring 17,000 feet. Above this layer a deck extends from about 18,000 to 23,000 feet.

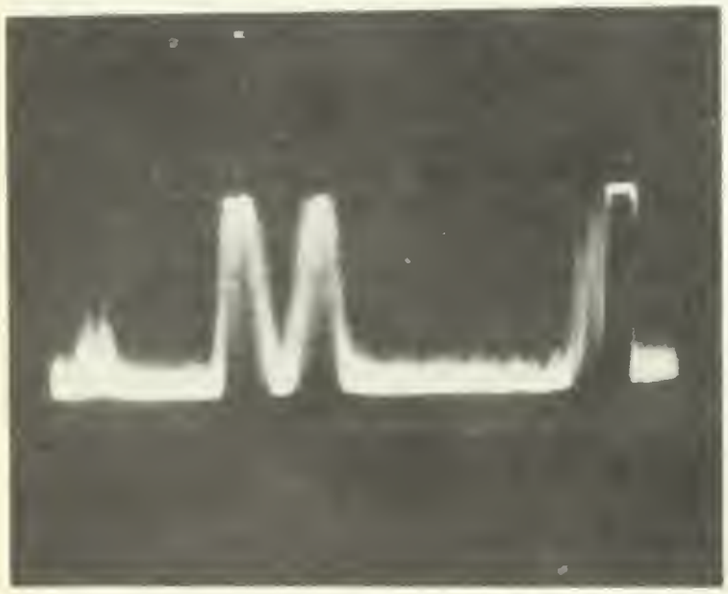


FIG. 76. —AN A-SCOPE CLOUD BASE AND TOP PRESENTATION.

The range markers are not shown in this picture. An overcast is indicated up to about 2,000 feet. The lower deck extends from 8,000 to 9,000 feet. Above is a cloud layer with the base at 10,000 feet and the top at 11,500 feet. A third layer is discernible from 14,000 to 15,000 feet.

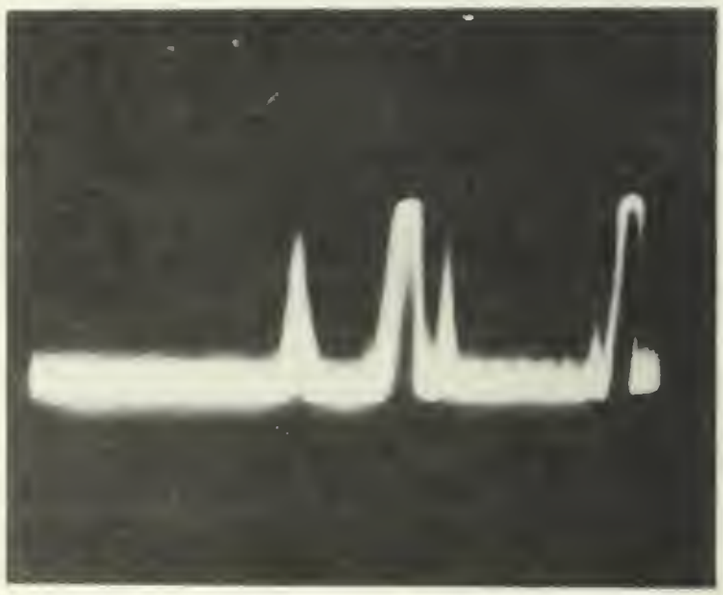
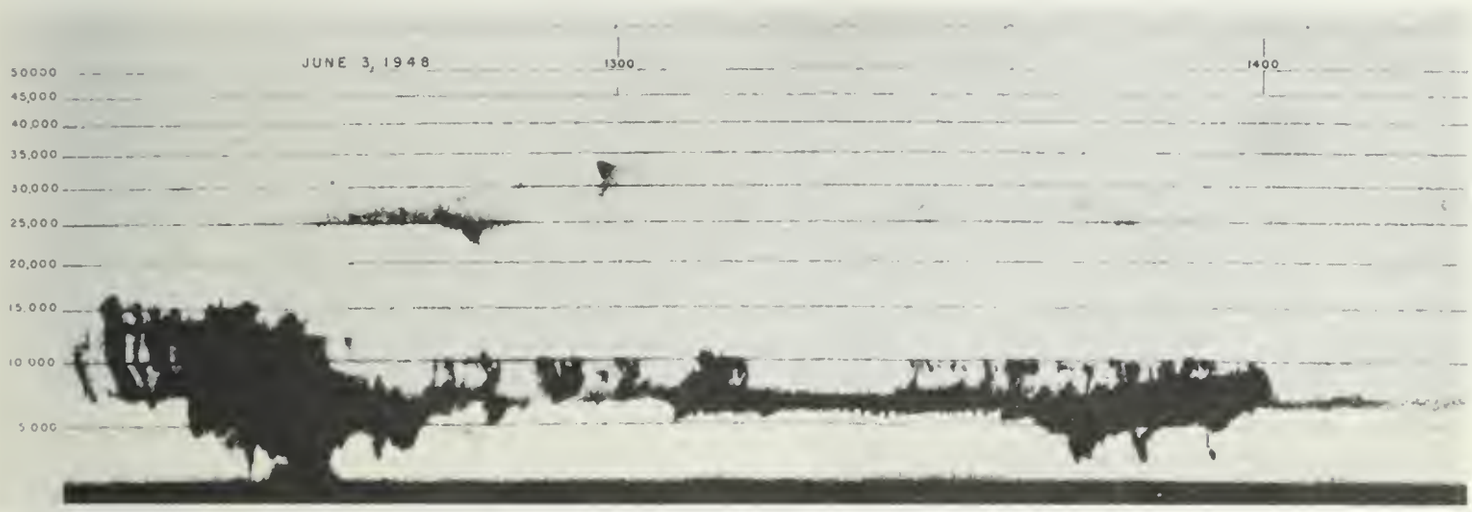


FIG. 77. —AN A-SCOPE CLOUD BASE AND TOP PRESENTATION.

In this picture a thin layer at 1,300 feet can be noted. A stronger return is evident at 6,000 feet. The third deck extends from 7,500 to about 9,000 feet. The top layer indicates a base at 12,000 feet and a top at 13,000 feet.



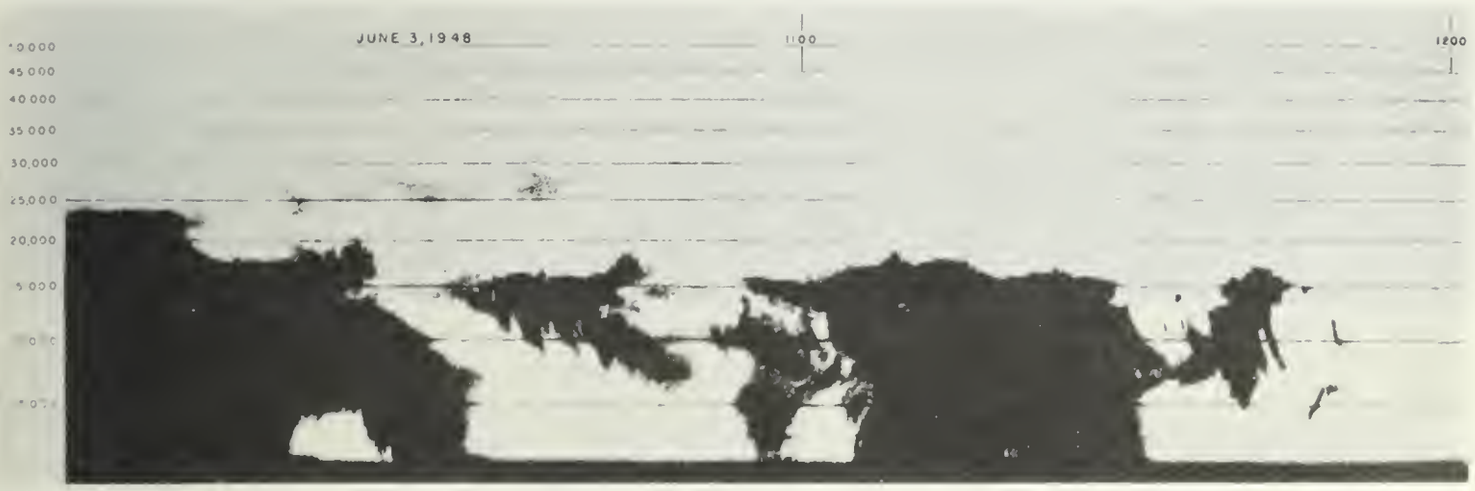


FIG. 78. —CLOUD BASE AND TOP RECORD.

This recording was obtained on 3 June 1948. The record starts at the bottom of the picture, at about 1000 hours. A storm is depicted, the top of which is approximately 24,000 feet. Shortly after the start of the record, a deck at 25,000 feet can be seen. As time progressed the storm dissipated; layers in shear effects may be noted.

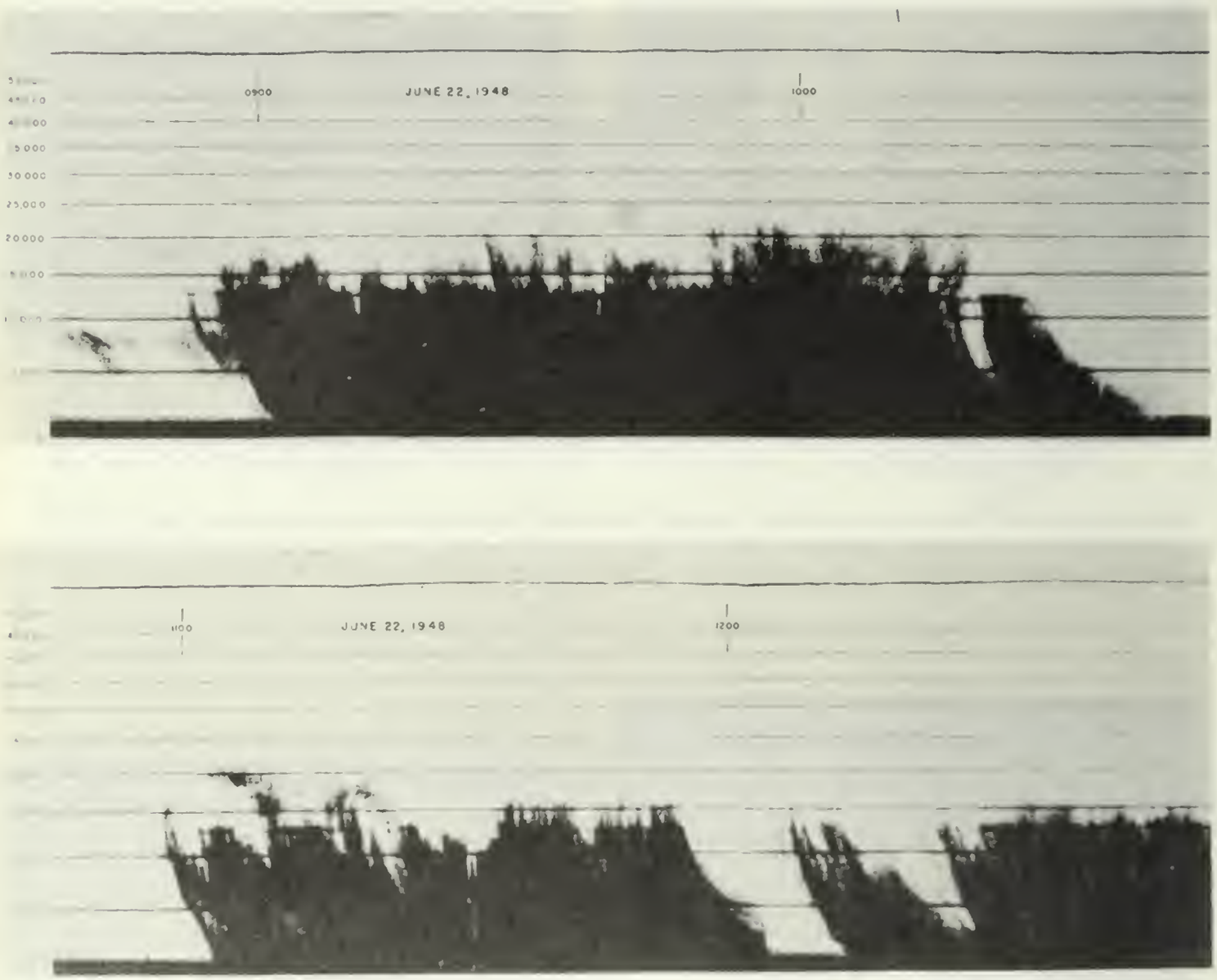


FIG. 79. —CLOUD BASE AND TOP RECORD.

This recording starts at the top of the picture in normal fashion. A bright line—the freezing level—is shown at approximately 13, 500 feet.

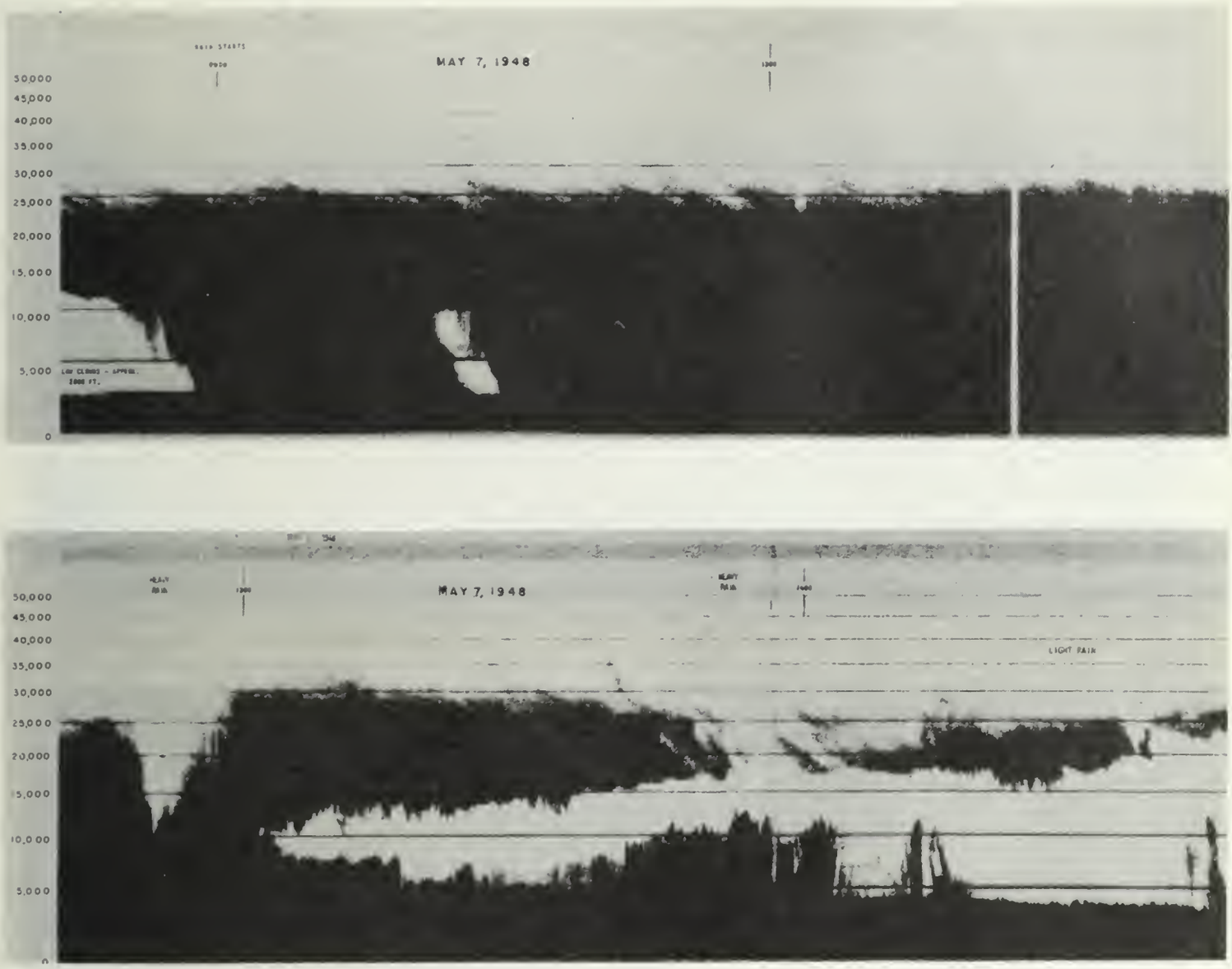
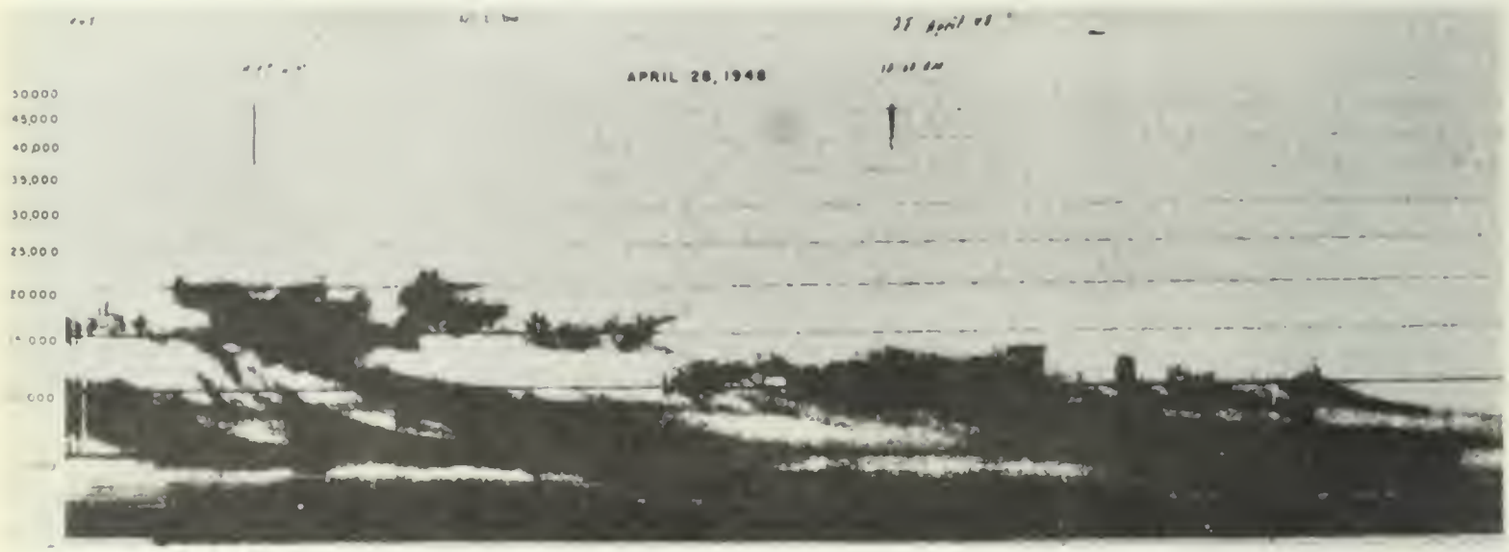


FIG. 80. —CLOUD BASE AND TOP RECORD.

A cross-section of a storm on 7 May 1948 is pictured. Rain started at about 0900 hours. Prior to this, low clouds are indicated at approximately 2,000 feet. At about time 1030 a blank space is noticeable in the recording, attributed to malfunctioning of the recording equipment for several minutes. Heavy rain started to fall, as indicated in the lower left-hand corner of the picture, and the loss in signals may be noted. During this interval it was impossible to receive indications from the top of the storm. There were two probable causes for the loss of signals, one being attenuation through a long path of rain, and the other the fact that the antenna paraboloid became filled with rain water.



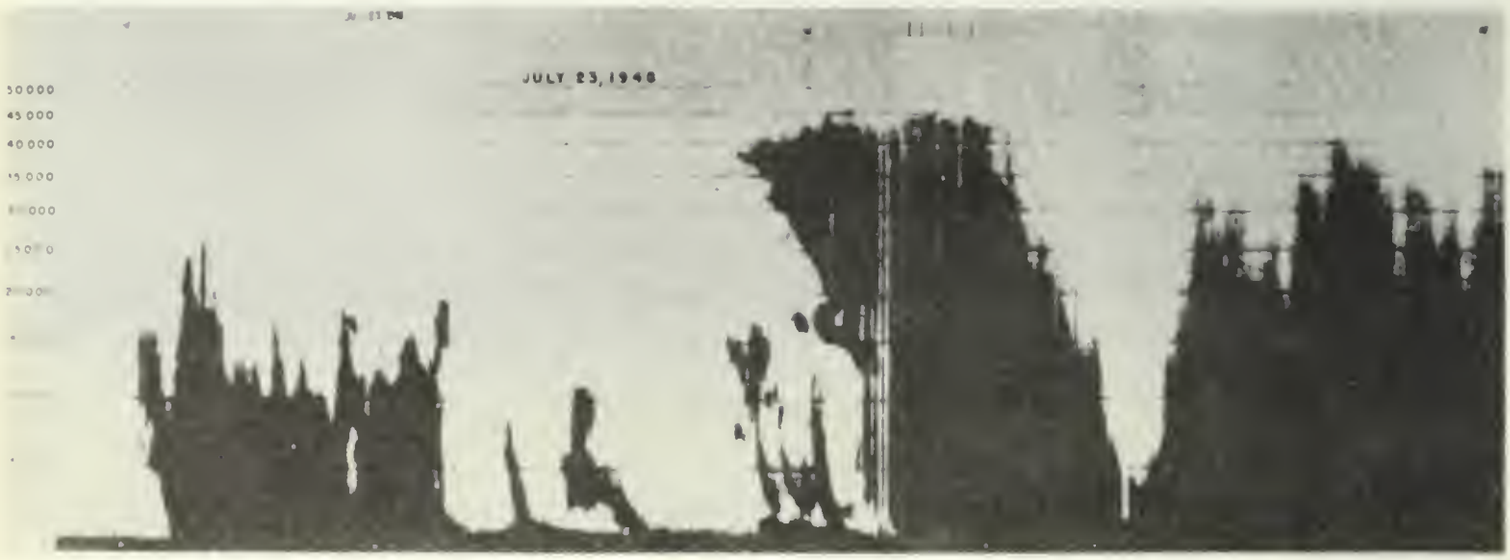


FIG. 81. —CLOUD BASE AND TOP RECORD.

Two different recordings are given here. The top bracket shows a peculiar formation of clouds which may be of interest to the meteorologist. The bottom bracket shows a thunderstorm of 23 July 1948 which built up to approximately 45,000 feet before very intense rain occurred. Note the very deep hole in the recording, caused by this intense rain, attributable either to attenuation or to water in the antenna dish and waveguide. The vertical white lines resulted from the set not being in tune, a satisfactory automatic-frequency control being nonexistent at that time.

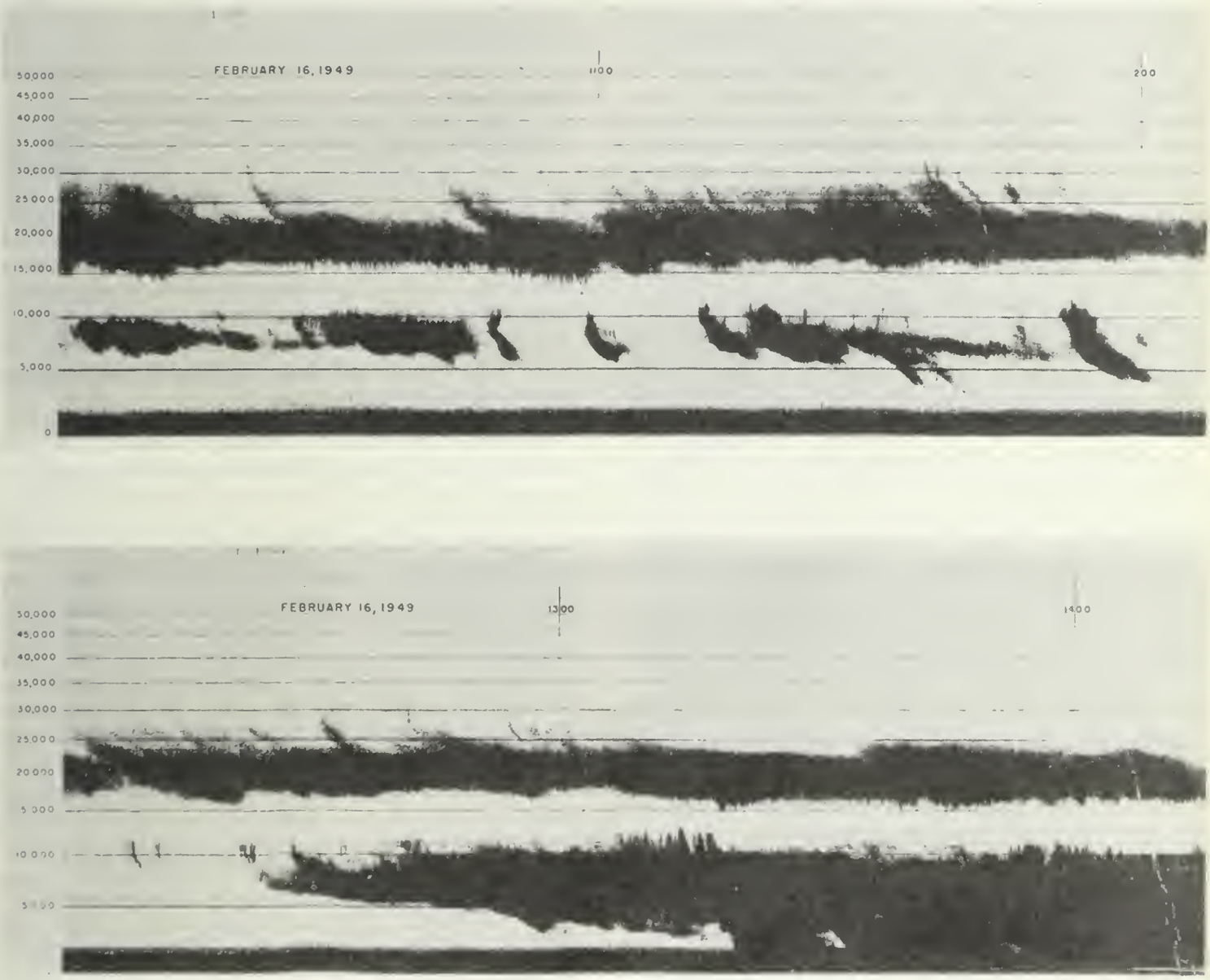


FIG. 82. —CLOUD BASE AND TOP RECORD.

Taken more recently, 16 February 1949, this picture presents a very good example of a recording as obtained from two decks. The cloud base and top indicator has been found to be a very interesting instrument. It is hoped that such equipment will, in time, prove of value to the meteorologist.

DISCUSSION

A. C. BEMIS. —Is there any discussion on this interesting gadget?

The Air Force is interested in an airborne model for use on their weather reconnaissance ships having antennas pointing both upward and downward from the airplane, so that they will be able to observe cloud layers both above and below them.

You did not say anything about "angels," Mr. Gould.

W. B. GOULD. —I have tried to stay clear of that subject, Alan, in what I have mentioned, since the causes of the "angel" phenomena are not well understood.

J. R. GERHARDT.[1]—The Electrical Engineering Research Laboratory has recently made a series of measurements which seem to support the existence of angels. These measurements involved direct recording of the fluctuations in the index of refraction of atmospheric air at microwave frequencies.

Modified for aircraft operation, the Crain refractometer has been flown at heights up to 10,000 feet mean sea level for locations over the New Jersey coastal area, Wright Field, Ohio, and immediately off the California coast. Fluctuations in refractive index are found at all measured altitudes, primarily in unstable atmospheres but also under stable conditions. While the root mean square values of these fluctuations are normally small and less than 1 part per million, individual variations of up to 5-10 parts per million have been found in apparently homogeneous air masses and up to 20-30 parts per million at certain air cloud interfaces and air mass boundaries. From a knowledge of the aircraft speeds and the periods of the observed fluctuations, the scale or size of these atmospheric discontinuities in refractive index are of the order of a few hundred feet.

While it has not been possible to obtain direct radar confirmations, the observed maximum index of refraction fluctuations, intensities and scales would certainly appear to be such as to permit a significant return of microwave radio energy.

(Editor's note. —A more detailed report on the index of refraction measurements is to be given at the January A. M. S. meeting.)

W. B. GOULD. —I think that is very interesting, and we are very glad to hear about these fluctuations because this cloud base and top indicator is such a beautiful angel detector.

[1]Assistant Director of Electrical Engineering Research Laboratory, University of Texas, Austin, Texas.

D. ATLAS. —We have been operating a 1 cm. radar very similar to the one which Mr. Gould described. Although our primary goal is the study of cloud physics, we have observed angels frequently. From the nature of these echoes and the associated meteorological conditions, I feel quite sure that they are not due to bugs but are atmospheric phenomena. For example, we have noticed the appearance of angels on occasion as an overcast breaks and the sun comes through. Although bugs like to flit around in the sun, it would be difficult for them to take off and get to an altitude of 3,000 to 4,000 feet in a few minutes after the clouds break. Also, angels have been detected just before and after rain showers, as well as during some of these showers. I doubt that bugs would be flying around under such conditions. Finally, on occasion we have observed angels in such abundance that the sky would have to be black with bugs if each "angel" were attributable to a bug.

D. D. REITER.[2]—One day, summer of 1949, I found strong angel returns appearing on the A-scope of the cloud base and top indicator at the Evans Signal Labs, and they were recording very well so I decided to stay all night. About sundown, the angels began to disappear. I went home for some lunch, and when I came back I found things were going fairly well except that the angels had begun to increase at an enormous rate. This went on, and at midnight the facsimile recording was still stronger and fuller than ever. The scope must have been saturated fully up to about 6,000 to 8,000 feet; however, there were no clouds in the sky. This condition remained for a few more hours, and about four o'clock the recording showed two layers of angels. They came down from scattered return at about 20,000 feet during the afternoon and evening to pronounced stratifications at about 3,000 or 4,000 feet. Two stratifications worked down to one, and it gradually died off toward noon the next day. I tried to find out if there was anything abnormal taking place in the weather and in the air body above, but the reports from the meteorology department said there was nothing abnormal at all—normal day, normal pressure, normal temperature, normal relative humidity—nothing abnormal.

D. M. SWINGLE. —On an afternoon like this, I would expect to see plenty of angels, and I imagine most of them would go to saturation, similar to clouds and any other type of radar signals.

(Editor's note. —It was a clear, warm day.)

[2]Electronic Scientist, Wright Air Development Center.

D. D. REITER. —There are strong radar returns in spite of the fact that there is nothing up there. You go out with a pair of field glasses or with a good telescope and you look up and visually one does not see these things.

W. B. GOULD. —I think Dave is a little confused at this point because I do not remember anything above something like 10,000 feet. I think that was the maximum. (Note by Mr. Reiter. — There are recordings remaining at Evans Signal Labs if definite proof is desired.)

A. C. BEMIS. —There is no other scheduled discussion of angels and the atmospheric discontinuities of this type on the rest of the program, so it might be well for those interested to air their views at this time. Are there any other comments?

M. L. STONE.[3] —I have observed angels on the ten-centimeter MTI (Moving Target Indicator). The cancellation was terrific, yet the angels kept coming in quite strongly. The radar was located on Deer Island in Boston Harbor.

A. C. BEMIS. —Can one say that angels are observed more frequently with a vertical pointing set than with one pointing horizontally? Or is it just more convenient to observe them in that direction?

W. B. GOULD. —I think that in our case it has been due to the fact that it is a by-product of the cloud base indicator where we were looking vertically and most of them have been seen while we were looking vertically. The other day, though, we went over to look at angels on the CPS-9, the first time I had ever seen them on a CPS-9. At that time, we tipped the antenna down so that we had more or less of a horizontal beam, and we saw more.

A. C. BEMIS. —What I was getting at was a clue as to whether these discontinuities tend to occur with stable stratification in the atmosphere, or whether just as often they occur with unstable turbulent elements.

R. C. JORGENSEN. —It is very seldom, if ever, that we set our antenna above the level of the horizon. Maybe it is because I do not live right, but I have never seen an angel.

(Editor's note. —It is a 10-cm. set.)

S. E. REYNOLDS.[4] —What is the possibility of seeing an angel on the inside of a cloud?

W. B. GOULD. —I think that the signal from the discontinuity producing this angel signal will be obscured by the cloud echo.

S. E. REYNOLDS. —You might not mistake an angel for precipitation in a cloud?

W. B. GOULD. —I think on three centimeters, it would be entirely masked. I might add that I have never seen an angel on 10 centimeters so that may tie in a bit.

A. C. BEMIS. —We have observed very strong echoes from lightning discharges. The ionized path of the lightning gives an echo which saturates the radar video even at great distance.

D. M. SWINGLE. —I might briefly summarize what we do know about angels. Insects and birds can certainly cause such echoes. However, insects form a rather flimsy explanation when one observes angels occurring at temperatures much below 60° F., while visual checks for birds have not supported that explanation in the cases most frequently seen. If these two causes are ruled out as the cause of most angel observations, the next most obvious cause would be discontinuities of dielectric constant or of its gradient occurring along surfaces in the atmosphere. The values of the dielectric gradient required are high, and the requirements on the smoothness of the surfaces rather severe, yet still within the realm of possibility. Mr. Gerhardt's comments on observed gradients of refractive index in the atmosphere are encouraging. The concentration of angels along inversions lends some support to this theory. One suspects that many angels may be caused by rapid changes in atmospheric dielectric constant, perhaps stirred up by turbulence.

D. ATLAS. —You were asking about detecting angels in clouds and storms. Well, it is possible to differentiate between angels and precipitation echoes by the fact that their signals are coherent and last a long period of time relative to a precipitation echo. Of course, it would be difficult to do this on a PPI scope. You would have to utilize the A-scope and watch the flutter characteristics of the echo. The angel echo makes a definite break in the A-scope base line and might last as long as a few seconds, while precipitation and cloud echoes flutter up and down very much like receiver noise. One must be a little cautious, however, since we have occasionally observed angels with flutter characteristics similar to those of precipitation echoes, and we had to go look out to make sure that it was not raining.

A. C. BEMIS. —Was that pointing horizontally?

[3]Electronic Engineer, Weather-Radar Project, Massachusetts Institute of Technology, Cambridge, Mass.

[4]Research Physicist, New Mexico School of Mines.

D. ATLAS. —Vertically.

A. C. BEMIS. —I was trying to picture them as an instability phenomenon, similar to "heat waves."

D. ATLAS. —I would like to say that these bubbles, strong return from a small volume, are extremely concentrated; they are very close together. The beam at the height of the level of these angels was perhaps 2,000 feet, that is perhaps 30 feet wide, so that it would have to be a very large number of angels per unit volume in order to give that flood, overlapping return in time and space, characteristic.

W. B. GOULD. —Mr. Atlas, you said something about the angels lasting for a very long period of time. I think the longest duration that we have ever observed has been something like ten seconds. That is very rare. I think it is more like two or three seconds and sometimes less than a second.

A. C. BEMIS. —Mr. Atlas means that the echoes on an A-scope look like airplane signals rather than precipitation signals.

D. ATLAS. —Yes, except that their durations are of the order noted by Mr. Gould. Cloud and precipitation echoes rarely persist in intensity for more than 0.01 second.

D. D. REITER. —The angel signals would fill the radar scope to a saturation up to 7,000 feet so it looked like rain. In this case, the rapid summation of return signals in the stratifications, or unusual strength, will go to saturation in the amplifier networks.

A. C. BEMIS. —This is an important subject in one respect. Since we do not know what these angels are as yet, we do not know how to use the information; but it appears to be meteorological in origin and therefore of interest to meteorologists.

UNIDENTIFIED.[5] —Can you explain why a wave length of 0.86 cm. was selected for the cloud set?

W. B. GOULD. —I can very briefly say that is based on theoretical work which was done by Ray Wexler. Ray wrote a memo in which he found that the shorter the wave length we went to, within limits, the better return we might get from the cloud particles, and I think he dodged the oxygen resonance band in that he wanted to stay away from this region and there were some other gases to avoid. At the time when this study was made the only equipment that was available was on a wave length of something like 0.86 cm., and that looked like a very good spot to locate in order to see cloud echoes.

Ray, could you elaborate any more on that study you made?

R. WEXLER. —It was two or three years since I made that study, but as I recall, in order to see the small drops, the signal return is inversely proportional to the fourth power of wave length. That means the smaller we go, of course, the bigger the signal, other factors remaining constant. But there were some attenuation bands that we wanted to stay away from—one at 1.23 cm. which is due to water vapor, another at 1.33 cm. There is another band that is an oxygen band at about 0.6 cm. If you get below 0.6 cm., there is another attenuation band which would have a maximum at about 0.2 cm. I do not think there was much to consider below about 0.7 cm. because there are too many bad attenuation bands. I chose 0.9 cm. as being a fair guess.

A. C. BEMIS. —Dr. Austin at M. I. T. made a similar study. Can you tell us anything about that, Polly?

P. M. AUSTIN. --I cannot say very much more than Ray has already said. I was studying the properties of a particular set whose wave length was already fixed. There is also the question of how much power can be produced efficiently at the shorter wave lengths; so the engineering details would have to be taken into consideration as well as the scattering and attenuation at different wave lengths.

M. G. H. LIGDA.[6] —I would like to point out just one thing and it is a little off the subject of the proper frequency of cloud detection equipment. I was quite struck, Bill, with your pictures (which I have not studied or examined carefully before). The distinct break in the base line of the sweep through the cloud echoes suggests to me that the fluctuation characteristics of the echoes from clouds may be significantly different from that of rain, where we very seldom see a distinct break in the base line. Occasionally, with echoes of saturation strength, the base line break may extend clear up to the saturation point. I would like to hear a few suggestions of what might cause this break, whether it be the pulse repetition frequency, or difference in motion of cloud drops and rain drops, and whether anybody thinks it is significant or not.

UNIDENTIFIED. —I think it is very significant to stable clouds, anyway.

[5]During the discussion there were comments by persons who were unidentified in the recording.

[6]Research Assistant, Weather-Radar Project, Massachusetts Institute of Technology, Cambridge, Mass.

W. B. GOULD. —I think that is a characteristic. I am a little afraid to jump off the deep end here and tell you what I think makes it that way.

D. ATLAS. —I do not think so. We have the same type of radar, of course, and have actually checked the flutter characteristics of cloud echoes. Surprisingly enough, the spectra of fluctuations as measured by Rasaph are quite broad, resembling those of rainfall. On the A-scope, however, when the receiver gain is increased, all the echoes will saturate and a distinct break will appear in the base line.

MONDAY EVENING SESSION
INTRODUCTORY REMARKS

BY H. R. BYERS

The session this evening is going to concern itself with the very practical problem of radar, and it happens to be the one in which the Water Survey here is the most interested and which also a number of you have done work on. I suppose the reason for my selection as chairman for this particular section is that I became extremely interested in this subject when I first started working with radar on the Thunderstorm Project in Florida. I very naively saw that it had great possibilities for the quantitative measurement of rainfall. Immediately, with the help of some of my colleagues, we started analyzing the data on the Thunderstorm Project in regard to some of the radar material vs. rainfall and came out with some results that indicated to us quite clearly that radar had possibilities for quantitative measurement of rainfall, particularly over small basins where small showers are important. I had yet to discover, however, the tremendous difficulties that were involved in the obtaining of the precise data, and it has been a process of education for me ever since, and I know that my education will continue this evening as we hear from various people who are going to talk about this subject.

When you get a bunch of radar engineers together, you are in sort of a dream world. It seems that they always talk about the radars of the future, the radars that are just barely on the drawing board. We who are concerned with the more practical aspects can not dream so far ahead in the future. We, however, like to think about the radars that are just going into production. This evening we have the opportunity of hearing about one of these radars — as a matter of fact, what appears to be the best radar for our purposes, namely the AN/CPS-9, and we are fortunate to have the man who is very deeply engaged in that development this evening, who, together with his colleague, has scheduled this paper. Details of the model AN/CPS-9 radar will be presented by E. L. Williams, Jr. and W. J. Schiff.

DETAILS OF THE PRODUCTION MODEL AN/CPS-9 STORM DETECTOR RADAR

BY WILLIAM J. SCHIFF[1] AND EDWIN L. WILLIAMS[2]

WITH DISCUSSION BY F. C. WHITE, E. L. WILLIAMS

Radar has been used recently as a tool to simplify the long and tedious job of gathering and correlating weather data. The AN/CPS-9 (Fig. 83) is a radar designed specifically for the purpose of gathering weather data. It features high transmitted power, narrow antenna beam width and high receiver sensitivity to insure return of echoes from precipitation targets within its range. The system includes an Indicator Console containing controls and indicators capable of presenting a three-dimensional picture of a precipitation system.

The AN/CPS-9(XE-1) was designed within the Signal Corps by engineers of the Evans Signal Laboratory. Three development models of the AN/CPS-9(XE-2) were produced by the Raytheon Manufacturing Company, based on the design of the XE-1 model. The outstanding advantages offered by the AN/CPS-9 are: its high-powered output which results in extended ranges, its better resolution because of the narrow beam width, its high receiver sensitivity, and its use of four cathode ray tube presentations which are capable of giving three dimensional displays of storms under surveillance, namely, range, azimuth and elevation.

The AN/CPS-9(XE-2) has been engineeringly tested at ESL (SCEI) and service tested by the Air Proving Ground (USAF). The equipment has been accepted and will be used as a radar storm detector by the Air Weather Service (USAF). Some refinements will be made in the production units as a result of tests conducted on the development models. Thus the equipment will be further improved as a radar storm detector.

The AN/CPS-9 production system retains the features of the developmental models and adds refinements and improvements found desirable during the operational testing period. The following features found in the developmental models are retained:

1. Peak transmitted power 250 KW min.
2. Pulse lengths of 0.5 and 5.0 usec.
3. Wave length 3.3 cm.
4. Antenna beam width 1° conical.
5. Continuous antenna scanning in azimuth or sector scanning in either azimuth or elevation. Also manual antenna control in either axis.
6. R. F. units mounted on antenna to eliminate r.f. rotating joints to provide maximum radiated and received power.

While retaining these features, each unit of the system has undergone design changes incorporating improvements suggested during field use of the system. The most striking changes take place

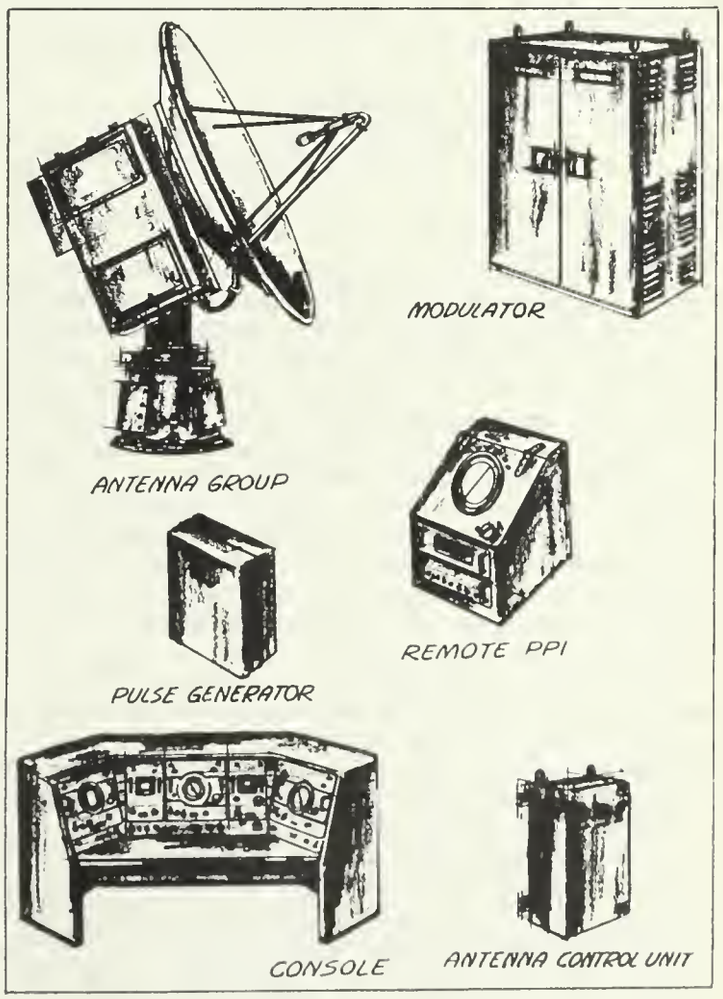


FIG. 83.—PRELIMINARY SKETCH OF AN/CPS-9 SYSTEM.

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in the antenna pedestal and the console. A picture of the over-all system will serve as a guide as we describe these units.

Antenna Pedestal. A completely new antenna pedestal is being designed with the following features:

1. 1000# weight (excluding weight of transmitter-receiver and dehydrator).
2. Larger base with 32" bolt circle for mounting.
3. A total of 48 slip rings, including 10 spares (for use with auxiliary equipment or for repair).
4. Rebalanced to eliminate need for wind balancing fins and spring used in the model.
5. The transmitter-receiver, which is mounted on the antenna, is being redesigned to provide space for an echo box. Also, the R. F. plumbing is being reworked to improve the accessibility and ease of changing the magnetron and duplexing tubes. An improved and simplified dehydrator will be mounted on the transmitter to eliminate the need for rotary air joints.

Indicator Console. The indicators are to be relocated and modified as follows (location from left to right):

1. 7-inch RHI. A 5" tube was formerly used.
2. 5-inch A-R scope.
3. 7-inch main PPI.
4. Indicator control (this is the ranging unit).
5. 7-inch off-center PPI (this replaces the "B" scope).

The console will also include the antenna scan control equipment and power supplies for the above units. A blower system is to be built in to exhaust the heated air from the console to a point outside the room in which it is located. The 7-inch indicators have been designed to use the same type of front panel bezel. Synchros and handcrank gearing are part of these bezels. Note that there is a built-in camera hanger on each of these indicators. A specially designed polaroid camera is being developed by the Signal Corps to be supplied with each equipment for recording scope indications. A jack is located on each panel to supply voltage for operating lights within the camera.

The main console PPI and the remote PPI have cam-operated microswitches built-in for synchronization of single frame movie type cameras. Two switches are provided, one at 0° and the other at 180° so the location of the film advance can be chosen to fit the storm pattern or local conditions.

The developmental models permitted detailed study of a small area through use of a precision B scope. Field use of the equipment showed that the B scope presentation was somewhat confusing and did not offer sufficient flexibility. The new system replaces the B scope with an off-center PPI capable of 2 radii off-center displacement. This scope permits enlarging the area desired for detailed study. 10° azimuth markers are available on this unit.

The two console PPI's provide four fixed ranges and three variable ranges. This added provision of variable ranges provides increased usefulness of the scope viewing area by permitting sweep expansion to most effectively display a storm pattern.

An elevation angle strobe generator is provided which causes the PPI displays to flash at an angle which is 5 times the elevation angle. This permits automatic recording of the elevation angle in PPI scope photos and it indicates elevation angle to the remote PPI operator.

The servo accuracy of all the PPI's has been improved to provide an indication with less than 1° of error. This is rather a small error when using a 1-speed synchro system.

The range scope now includes two additional A type sweeps of 75- and 400-mile ranges. The R sweep circuits have been modified to present the range strobe on the 5-mile sweep as well as on the 20 and to permit ranging with the strobe down to 1/2 mile when the system is operated on long pulse. (On the model, the minimum range was about 12 miles on long pulse.)

The console layout has been carefully studied to improve the location of operating controls. All controls not normally used are now located back of panel. All controls common to two or more units are located in the same relative position on the units, i.e., range switches are always in the lower left corner. Lighting will be provided for illumination of the control labeling.

An item of interest to some of you is that all 6J6 tubes have been eliminated from the design. This tube was the only miniature twin triode available at the time the developmental models were built. The use of this tube turned into a field maintenance headache. All circuits have been checked to use newer type tubes to eliminate this problem.

A telephone switchboard is provided to permit communications between the remote telephone stations at the antenna, modulator, remote PPI, and the console.

Remote PPI. Redesign on this unit is primarily of the nature which will make it easier to produce. A blower is being added and the PPI servo amplifier circuit is being changed to use the same servo being designed for the console indicators.

Pulse Generator. Circuit design has been completely checked for use of 12AT7 tubes in place of 6J6's. However, this has resulted in practically no circuit changes. General appearance and layout of the unit will be unchanged.

Antenna Control Unit. This unit was formerly made up mostly of synchro balancing capacitors and a few control circuits. The new unit will contain the electronic servo amplifiers for controlling the antenna hydraulic servo. In addition to the servo amplifiers, the antenna safety switch and a telephone station are located at the antenna control unit.

DISCUSSION

F. C. WHITE.*—Would Mr. Williams mind giving us a little information on what the rainfall height and rate has to be to detect a storm with a 400-mile range with this particular radar?

E. L. WILLIAMS. —The rain would have to be at a height of 80,000 feet to be detected at 400 miles under normal atmospheric conditions. Since this is an extremely rare occurrence, storm detection at this maximum range will probably only occur under conditions which cause the beam to bend. Of course, this also depends on the siting of the antenna. Any rain which occurred at this altitude would probably give a return on the radar since only 1/100 of the beam need be filled with light rain to give a detectable signal at this range. The beam width is 37,000 feet at this range.

*Air Transport Association.

COMPARISON OF AVERAGE RADAR SIGNAL INTENSITY WITH RAINFALL RATE

BY PAULINE M. AUSTIN*

I think that all of us who have seen precipitation echoes on the various radar scopes and noted the intricate patterns which they form, are aware of the advantages in coverage which measurement of rainfall by means of radar would have as compared with a network of rain gages. While the rain gages can record the rainfall only at certain fixed points which are usually far apart, a single radar installation can see all the precipitation within several thousand square miles. Therefore, I will not dwell upon the desirability of such measurements, but will launch immediately upon a discussion of some of the problems which Dr. Byers has mentioned so cheerfully and then left for us to cope with.

In the first place, the measurement of the radar signal intensity is difficult in the case of precipitation echoes because of the rapid fluctuations which occur. It is necessary to devise some method for averaging the rapidly varying time function which represents the rain echo. Some observers have attempted to do this averaging visually by choosing an average level for the lacy structure which appears on an R-scope. At the Weather Radar Project this averaging is done electronically by an instrument called the Pulse Integrator.

Once the average signal intensity has been measured, further difficulty is encountered because the relation between the precipitation rate and the average signal intensity is not unique. The average radar signal intensity depends upon the size and number of raindrops in the illuminated volume of air. Hence, a rainfall whose intensity is 5 mm./hr. and which consists of relatively few large drops would produce a very different radar return from that of rain of the same intensity, 5 mm./hr., which consists of a very large number of rather small drops. Calculations have been made, however, for the average radar signal intensity as a function of precipitation rate by assuming an average drop size distribution for each rainfall rate. This was done a number of years ago by Dr. Ryde, using drop size distributions as measured by Laws and Parsons.

Some experimental work has also been done in this field. Several investigators have made simultaneous measurements of rainfall rate and radar signal intensity. In some cases measurements have been relative in value, that is, the radar measurements have consisted of observing the ratio between transmitted and received power or the signal-to-noise ratio for various rainfall rates. There have also been some measurements of the absolute intensities of radar echoes from precipitation, made by calibrating the system by means of a signal generator. In general, the increase of radar signal intensity as a function of rainfall rate was found to agree at least roughly with the relation predicted by Ryde's calculations.

As mentioned previously, the work done at the Weather Radar Project differs from that of other investigators chiefly in the method used for averaging the signal. This is done by the Pulse Integrator, which averages the voltage from each separate radar pulse over a period of about one second. This average signal intensity is recorded on an Esterline-Angus recorder and is compared with signals of known strength from a signal generator. We obtain the measurements by pointing the radar beam over a rain gage, which is situated about 12 miles away from the radar site, and making simultaneous recordings of the precipitation rate and of the average signal intensity. The radar set used is an SCR-615B with a wave length of 10 cm. and a beam width of 3° between half power points.

The first measurements of this type were made in the summer of 1949, and we were rather dismayed to discover that the observed radar intensities were very, very much less than those predicted by the calculations, a matter of 14 or 15 decibels. We decided that a careful calibration and checking of the whole radar system should be made to see if there were power losses of which we were unaware. We were further spurred on in this work by the publication last year of a paper by Hooper and Kippax, who had made simultaneous measurements of precipitation rate and radar signal intensity as observed by a vertically pointing radar. They had calibrated their radar by measurements on a standard target, a metal sphere. Their results showed very good agreement, in both relative and absolute values, with Ryde's calculations. We followed their example and obtained some aluminum spheres, 20.75 inches in diameter, as standard targets. Fig. 84 shows the results of measurements on two of these spheres which were carried aloft by balloons and

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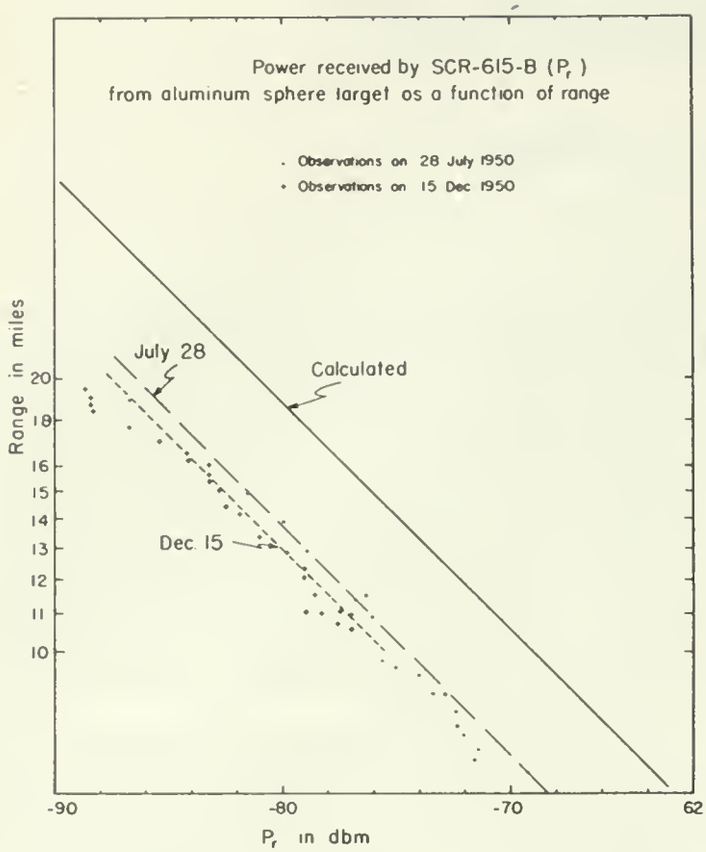


FIG. 84.

tracked by the radar. The intensity of the radar signal received from the sphere, or any point target, depends upon the power transmitted by the radar, the range of the target, the radar scattering cross-section of the target and the over-all gain of the system. The solid line shows the calculated value of the received power, in decibels with respect to a milliwatt, as a function of the range of the target for our particular radar set and metal spheres. The dots and crosses indicate the observations for the two separate calibrations. We find that the observed values are considerably below the calculated value, 5.5 db for one of the runs and 6.5 db for the other. The difference of one decibel between the two runs we attribute to the fact that a new magnetron was installed in October, 1950. A very small difference in the frequency of the output of the two magnetrons causes a difference of one decibel in the antenna response. As a result of the measurements, we believe that somewhere in the system there is a loss of about 6 db of which we had previously been unaware.

While these experiments were being conducted, we noticed a target with an exceptionally steady signal located very close to the point where the sphere was released. Many measurements have been made on this target, which turned out to be a pair of water towers, and the readings of the power received from it can be repeated to within half of a decibel or less. This target provides a convenient means of making a day-to-day check on the system. It has been encouraging to observe a high degree of stability from day to day.

Having made this calibration with the standard target, we decided that the next step should be the consideration of quantities which are involved in observing precipitation but do not enter into the measurement of point targets. The first difference lies in the fact that the target is spread out over the whole cross-sectional area of the beam instead of being confined to the point of greatest intensity at the center of the beam. Hence not only the maximum gain of the antenna is involved but the whole beam intensity pattern. We used the signal from the water towers to measure the beam pattern. The antenna scanned them in both the horizontal and vertical directions, and in Fig. 85 is shown the illumination pattern which was obtained. The little crosses and dots show where two separate measurements were made; it can be seen that they agree very closely.

In the calculations pertaining to the intensity

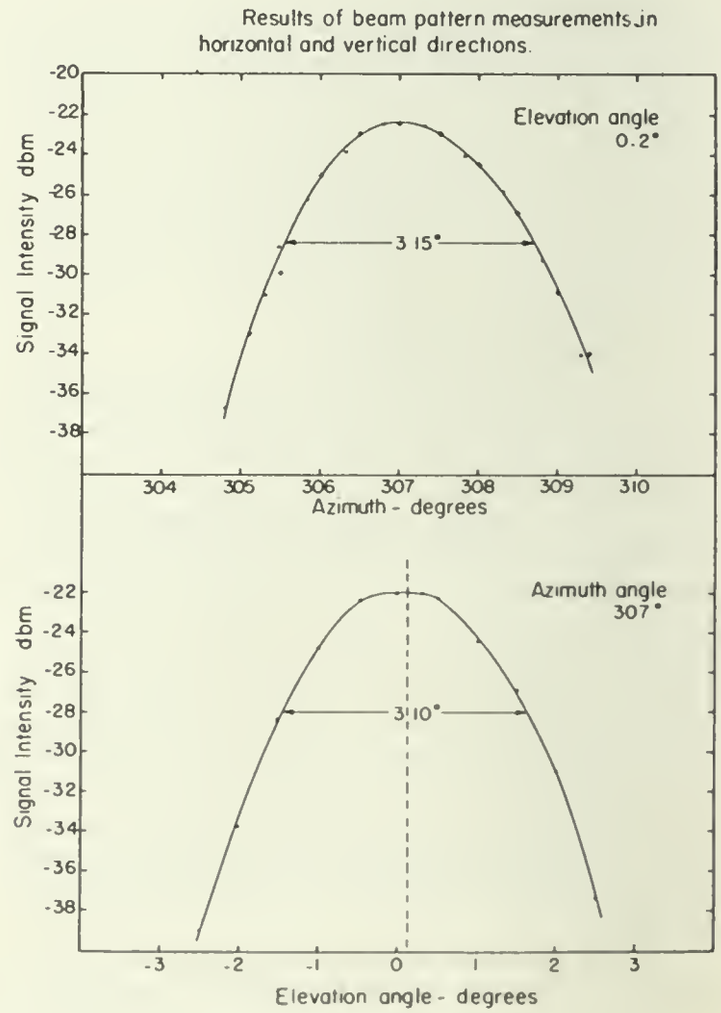


FIG. 85.

of the radar signal from precipitation, we assumed the beam to be conical, 3° in aperture, and that the illumination is uniform from the center of the beam out to an angle of 1.5^{o} and then drops sharply to zero. Calculations made for the power received from precipitation illuminated by a beam having the measured pattern showed that the total power is about 83% of that for a straight-edged 3° beam having the same intensity at the center. This difference amounts to about one decibel.

The measured beam pattern was also used to calculate the amount of energy which is lost below the horizon whenever the beam is at a low elevation angle. Any measurement which we have made with an elevation angle of one degree or less has been corrected for the amount of energy lost below the horizon.

Another difference between the measurements made on the standard target and those made on precipitation lies in the averaging process of the Pulse Integrator. The signal from the sphere is steady, and we get the same voltage from each pulse, but for the fluctuating signal from the rain the Pulse Integrator must average pulses which vary greatly in amplitude. Moreover, calculations for the average received power are based on the assumption that the power of a number of pulses is averaged. Actually, the Pulse Integrator averages the voltage, which is proportional to the square root of the power. If the rain drops are in random positions in space, it can be shown that the error involved in averaging the voltage instead of the power is one decibel. Another error which might be introduced by the rapid variations in the signal is distortion caused by nonlinearities in the detecting and receiving system. We have measured the response of the system and find that it is linear over most of the range at which our rain measurements have been made. There is no appreciable error due to nonlinearity in the receiving and detecting system.

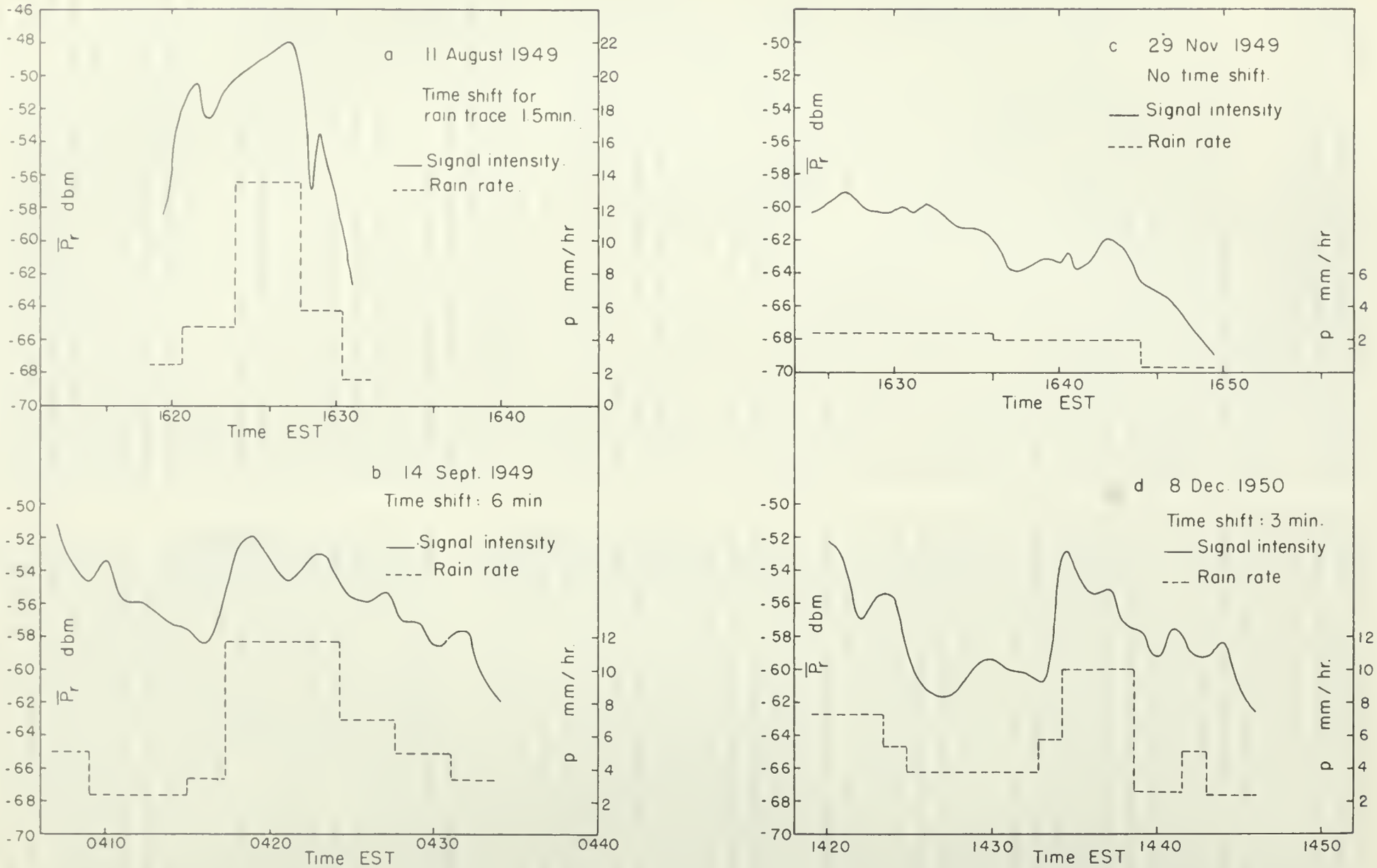
Another factor which might affect rain measurements but not the sphere measurements is attenuation either by rain along the path or by water on the radome. Attenuation by rain at a wave length of 10 cm. is negligible. To test the effect of water on the radome we sprayed it while making measurements on the standard targets. It turned out that there was no diminution of signal intensity except for a very heavy spray, in which case the signal decreased as much as two decibels. None of the measurements discussed here have been made under conditions of heavy rain at the radar site; therefore, no correction has been made for this effect.

The rain gage used in these measurements is a Fergusson gage which measures cumulative rainfall amounts. We have put on an extra large funnel, which increases the sensitivity tenfold. Rainfall amounts can be read to about one thousandth of an inch and the time to about one half of a minute. However, the uncertainties in these readings are fairly large because of the thickness of the trace. We have averaged the rainfall rates over intervals of at least two minutes, since a shorter time did not seem feasible because of inaccuracies in reading the rain trace.

The data are plotted as shown in Fig. 86. The average received power, $\overline{P}_r$, in decibels below a milliwatt, and the rainfall rate, p, in millimeters per hour, are plotted against time. The received power has been normalized to a range of one mile. In each case the time scale for the rain gage trace has been shifted by a few minutes in order to bring the peaks of the two curves to the same position. There are two factors which make such a time shift necessary. In the first place, the energy scattered to the radar comes from a region one or two thousand feet above the rain gage. Since showers are usually sloping because of higher wind velocities aloft, the strongest part of the shower usually reaches the region illuminated by the radar a little before the time when the maximum rain rate is recorded on the gage. In the second place, the absolute values of time on the rain gage trace may be in error by one or two minutes. The runs in this figure were chosen to illustrate different types of rainstorms. The data taken on August 11, 1949, are from a strong convective shower. On September 14, 1949, there were also convective showers, but slightly less intense and occurring in a region of more general and widespread rain. On November 29, 1949, there was light steady rain associated with a low pressure area located off the Atlantic coast. On December 8, 1950, moderate rain extended over a large area and lasted for several hours.

After the data had been plotted on graphs like this, corresponding values of signal intensity and rainfall rate were read off and plotted as shown in Fig. 87. Times when the signal intensity was changing rapidly were discarded because corresponding measurements on the rain gage were not available. Measurements were taken on 15 days. There were 25 runs totaling about 13 hours. On August 29, 1949, RHI photographs showed that the radar echoes were coming from a layer several thousand feet above the ground. Apparently most of the rain was evaporating before reaching the ground. Because of these abnormal conditions the data for this day were discarded.

The solid line in Fig. 87 shows the calculated relation between $\overline{P}_r$ and p and includes all the corrections indicated by the various calibrations which have been discussed. The dashed lines were drawn to show what appears to be the best "average" relation between $\overline{P}_r$ and p. The slope of the lower part agrees well with the slope of the calculated curve, but there is a difference of 7 db in the absolute values. The slope of the upper portion of



Examples of data showing simultaneous measurements of radar signal intensity, $\overline{P}_r$, and rain rate, p.

FIG. 86.

the dashed line approximates a linear relationship between signal intensity and rain rate. Such a relationship would exist if the drop size distribution remained constant, that is, if the number of drops of all sizes increased proportionately as the rainfall rate increased. In this case the heavier rain would not contain larger drops than the lighter rain, but simply more of the same sizes. When the signal intensity increases more rapidly than the rainfall rate, as in the lower portion of the dashed curve, the indication is that larger drops do occur in the heavier rain.

From this graph we are left with two things to consider. One is that the measured signal intensities fall well below the calculated values. The other is that the spread in signal intensity for any given rain rate is quite broad. Therefore, in order to obtain accurate measurements of precipitation rate, we would need more information than simply the average radar signal intensity. We would have to know whether we were observing a storm which was apt to have drops which are bigger than average or whether it was a storm which was apt to have relatively small drops. We would therefore require an understanding of the differences or likenesses which exist from one storm to another. Fig. 88 shows what we have tried to do in considering the differences or similarities from storm to storm. Each of these graphs shows the data from a single storm. As before we have plotted the average signal intensity in dbm. against the rainfall rate. In each case the dashed lines indicate the slope for a linear relationship between the two, which would suggest a constant drop size distribution. From this group of measurements it seems that rain, whether light or heavy, in the same shower or the same storm tends to have more or less the same constitution. These indications are included because they seem to show interesting trends. However, the number and accuracy of the data are not sufficient to warrant drawing definite conclusions. Figures c and d show data from two convective showers, and the numbers indicate the chronological order of the measurements. It appears that the drops are much larger in the early part of each shower. This is not very surprising since we often see very large drops at the beginning of a thunderstorm. I only mention it to show that such individual characteristics should be taken into consideration in any attempt to make precise measurement of rainfall rate by means of radar.

We now consider those seven decibels that separate the observed and calculated values of the radar signal intensity. There are several factors which may contribute to them and which I will mention briefly. First, however, I would like to refer to the measurements made by Hooper and Kippax, who obtained very good agreement between theory and observations. We have considered very carefully their method of averaging, which is to match the height of a calibrated signal from a signal generator with the height of the rain echo on the R-scope. Now the rain echo is a lacy and mobile structure so that its height on the R-scope is not clearly defined. The level which was matched to the calibrated signal was described as the maximum level to which the signal rose for about 95% of the time during a period of about half a minute. As nearly as we have been able to determine, such a method would give a reading about 5 to 7 db above the value obtained by the Pulse Integrator. It seems, therefore, that the results presented in this study do not in reality disagree with those of Hooper and Kippax, but that both sets of observations indicate lower radar intensities than would be expected from theoretical considerations.

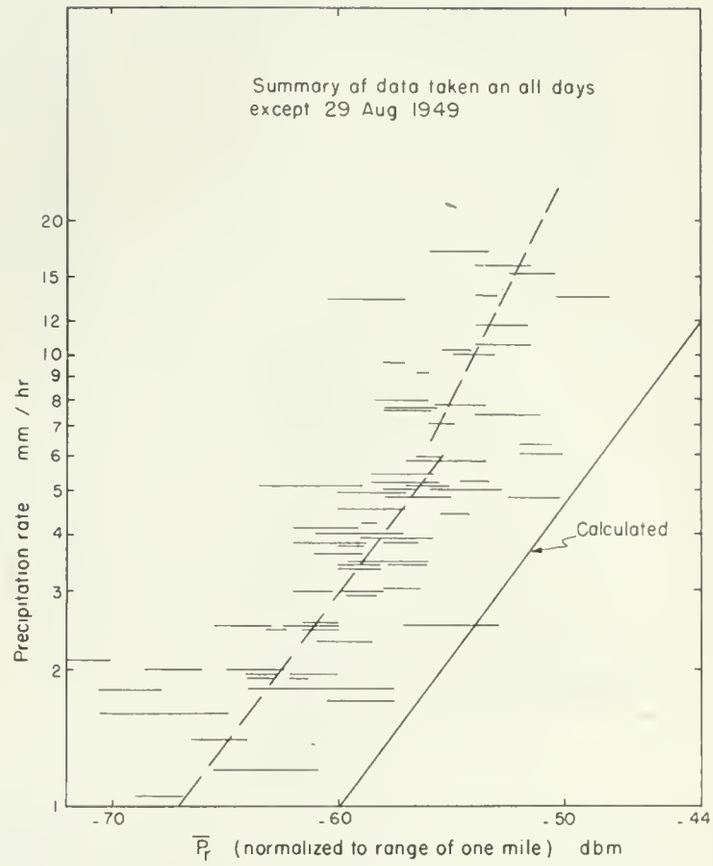
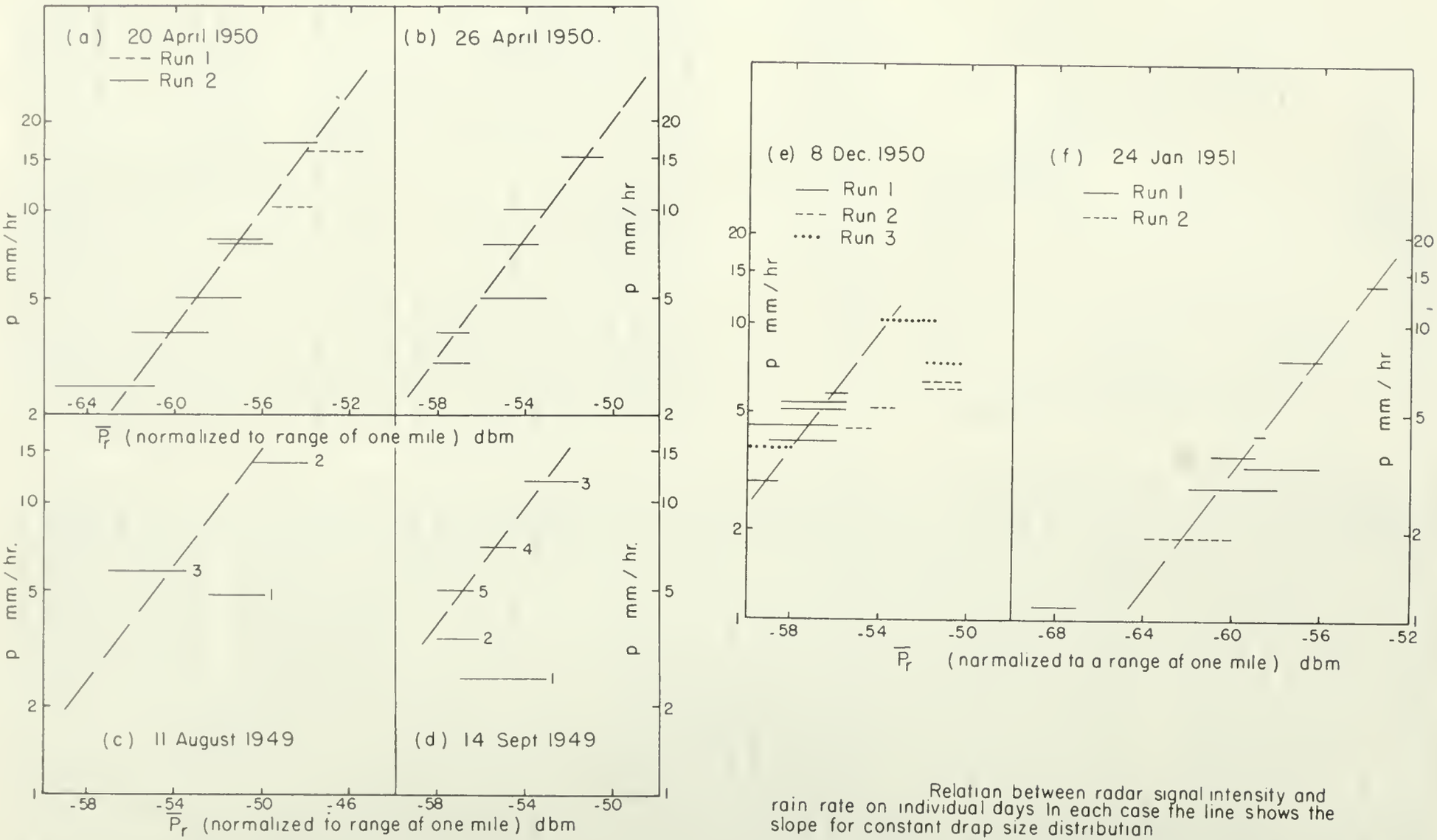


FIG. 87,

I will now mention several factors which might contribute to the observed discrepancy. One is the drop size distribution. Perhaps our rain does not average to the same drop size distribution as that measured by Laws and Parsons. We made a few drop size measurements of our own on four days. On two of those days the reflectivity as calculated from our data was very close to that obtained from their data. On the other two days our values were much lower, but on the same two days the observed radar reflectivities were also much lower than aver-



Relation between radar signal intensity and rain rate on individual days In each case the line shows the slope for constant drop size distribution

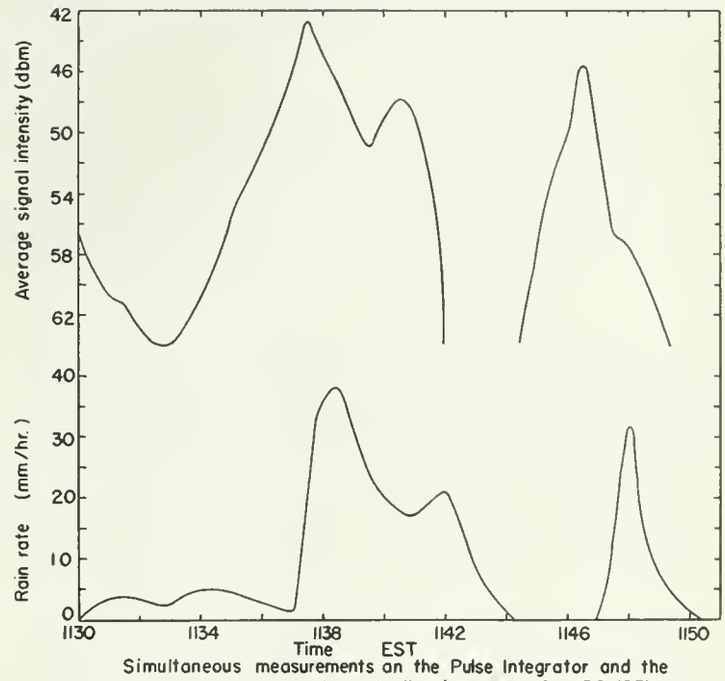
FIG. 88.

age. Hence it seems that differences in drop size distribution can explain the broad spread of the measurements on either side of the average line, but do not account for an appreciable portion of the discrepancy between the calculated and observed average values.

Another factor which might contribute to the low readings is the possible growth of rain drops between the level one or two thousand feet above the ground, where the radar power is scattered, and the ground, where the rainfall is observed. We have made some calculations for the possible contribution of this factor. It appears that under ordinary circumstances the contribution would be negligible. A rain drop one and a half or two millimeters in diameter would not grow enough in falling through 1,000 feet of thick cloud to add more than a fraction of a decibel to the scattered power. We do have some observations taken on a day when the antenna was elevated three degrees and some very small rain drops fell through 3,000 feet of thick cloud, between the regions where the radar and rain gage measurements were made. I think in this one case the growth of the drops may have introduced an error of 4 or 5 db, but in general the effect of this factor should be negligible or at least quite small.

Another factor which might contribute to the discrepancy is the value for fall velocities of rain drops which are used in the calculations. The fall velocities used are those measured in still air. However, if there is a down draft in the rain, then there would be an apparent increase in rainfall rate as compared with rain falling through still air. We have not enough information about the existence and velocities of down drafts to make an accurate assessment of this effect. But it seems that except in the case of very small drops, downward velocities of several meters per second would be required to introduce an error of more than one decibel.

Another factor which has been mentioned is the question of whether or not our three-degree beam is filled with precipitation at a range of 12 miles. There might be cases where the shower is in the middle of the beam but does not extend to the edges. At a range of 12 miles a storm a mile across would fill the beam. Moreover, the fact that we have averaged the rain intensities over a period of two minutes should take care of brief but intense



Simultaneous measurements on the Pulse Integrator and the Hudson-Jardi rain gauge. Aug. 20, 1951

FIG. 89.

portions of the storm. Hence we believe that even though this factor may, on occasion, contribute a little to the discrepancy, it cannot account for very much.

We hope to get more detailed information on this whole problem with the use of a Pulse Integrator on our CPS-9 radar, which has a 3 cm. wave length and a much narrower beam. We also have a new Hudson-Jardi type rain gage which measures rainfall rates directly and is much more sensitive than the Fergusson gage. Fig. 89 shows a sample of the data which can be obtained with the new rain gage. In this figure the average signal intensity and the rainfall rate are plotted against time. There is a slight lag between the peaks of the two curves since no time shift has been applied. The very beautiful agreement between the shapes of the two curves can be noted. One shower was scarcely longer than two minutes in its total duration; yet the details can be observed on the new rain gage, whereas the whole shower would have been averaged together on the old one. We hope that more detailed information will soon be available with this new experimental setup.

QUANTITATIVE RADAR-RAINFALL PROBLEMS

BY GERALD FARNSWORTH*

WITH DISCUSSIONS BY R. WEXLER, S. E. REYNOLDS, D. D. REITER, H. R. BYERS, AND G. W. FARNSWORTH

INTRODUCTION

The objective of the Illinois State Water Survey radar program is to establish, experimentally, the relationship between received power and rainfall intensity, and at the same time develop a technique that will provide and record information on intensities and total rainfall over an area such as a watershed. In working toward this objective, numerous problems and questions have arisen. Some have been solved, but a major portion of them are still open issues.

The characteristics of the radar set need to be known quite accurately. Beam width, pulse length, transmitter power output, and receiver sensitivity are some of the most important. Some of the important meteorological factors are rainfall intensity, drop size variation and distribution as related to intensity, and storm size.

RADAR PERFORMANCE MEASUREMENTS

The electrical characteristics of a radar set must be accurately measured to use it in determining rainfall intensities. The two parameters that seem to be most unstable are transmitter power output, and receiver sensitivity. Transmitter pulse length and beam width may vary in special cases but generally can be assumed as constant. A one decibel change in either transmitter power or receiver sensitivity will produce approximately a 12 per cent change in calculated rainfall intensity. Thus it is important to know the absolute value of transmitted power and receiver sensitivity accurately.

The only equipment the Water Survey has had available for checking transmitted power and receiver sensitivity are war surplus signal generators. The TS-13 and TS-263 operate in the 3 cm. range, and the TS-155 operates in the 10 cm. range. These sets are far from satisfactory. It is doubtful that the absolute accuracies are better than plus or minus 3 db, although relative accuracies of a few tenths of a db can be obtained. The calibrated attenuator dials are not finely divided, so interpolation is necessary to make readings closer than one db. Even if the dials did have fine divisions, accuracy would be poor, for the calibration will vary with the frequency. The specifications for a 9,000 mc. f-m set built by Sperry, the commercial equivalent of the TS-147, specifies a watt meter accuracy of only ±1.5 db, using the calibration curve supplied with the instrument.

The same difficulties are found in receiver sensitivity measurements. The technique we use in obtaining data requires that received power to produce a barely visible signal on the PPI be known. This "threshold of visibility" method introduces another important radar factor, the PPI brilliance setting. The brilliance must remain constant, or the received power will vary for threshold of visibility.

The manner in which the signal generator is connected to the wave guide directional coupler is also important. Experiments were conducted last spring to determine how RG-8U and RG-9U flexible coaxial cable functioned at 9,000 mc. The attenuation of RG-8U was much greater than for RG-9U, but even more important was the change in attenuation introduced when the cable was bent or twisted. As much as 0.3 db/ft. change was noted with RG-9U cable. The RG-8U had even greater change. If flexible coaxial cable is used with the test set, it should be kept to very short lengths. It would be much more desirable to use flexible wave guide.

EFFECT OF BEAM WIDTH

The antenna beam width is quite important in quantitative measurements. The simplest relationship of received power to rain intensity is based upon the assumption that the rainstorm completely fills the beam to the half-power points. Corrections can be made if the beam is not completely filled, but they are complicated and may introduce considerable error (1).** The useful range of the radar is limited to that in which its beam is completely filled, unless extensive corrections are made.

The Thunderstorm Project (2) found that the initial radar echo usually had a height close to the

*Electrical Engineer, Illinois State Water Survey.

**Numerals in parentheses are reference numbers in BIBLIOGRAPHY at end of this paper.

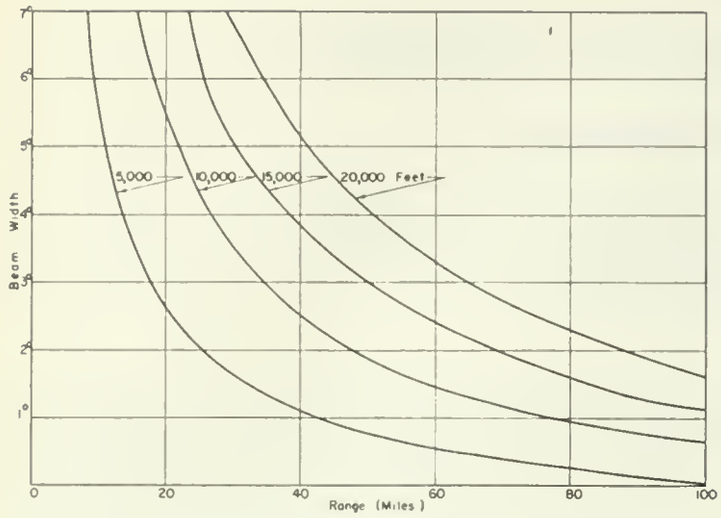


FIG. 90. —UPPER HALF-POWER POINT OF BEAM AS A FUNCTION OF BEAM WIDTH AND RANGE.

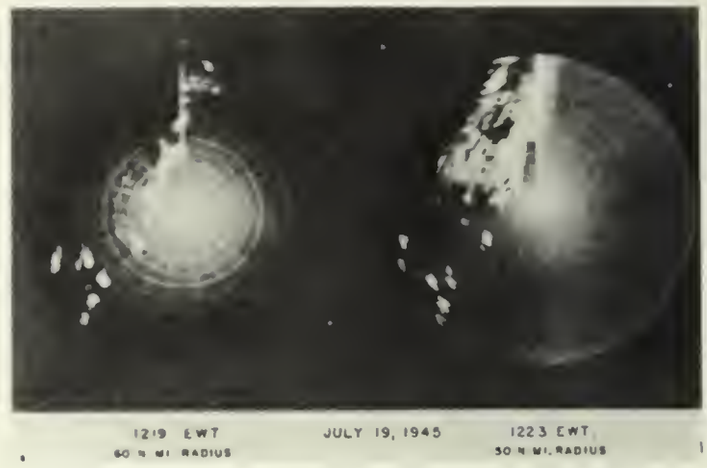


FIG. 91. —APQ-13 AND 584 SCOPE PICTURES SHOWING SHADOW BEHIND STORM.

freezing level although the thunderstorms grew to heights of over 25,000 feet. Therefore, it is assumed that the useful range of the radar for quantitative measurement is reached when the upper half-power point of the beam reaches the freezing level. Fig. 90 is a graph showing how the freezing levels and beam widths will affect the useful range. An earth's effective radius of 4/3 actual radius has been used. During summertime thunderstorms, the freezing level is in the vicinity of 15,000 feet. With the three-degree beam width of the APS-15, our 3 cm. radar, the useful range is 50 miles. The TPL, a 10 cm. set, with a beam width of 7 degrees has a useful range of about 24 miles.

During late fall and early spring, warm front rains, the freezing level is considerably lower, 8,000 feet or less. This would greatly reduce the useful ranges. Even with a 10,000 foot level the useful ranges are reduced to 35 miles for the 3-degree beam width and 18 miles for a 7-degree beam width.

These same limitations will apply to storms of small diameter. The Thunderstorm Project obtained a mean horizontal cross-sectional area of about 10 square miles from 115 thunderstorms. Assuming the storms were circular, the mean diameter would be 3.5 miles. This would impose about the same range restrictions as the freezing level.

CORRECTION OF VIDEO CIRCUITS

Modifying the video circuits of the APS-15 has greatly improved its use for quantitative rainfall measurements. Close observation of the PPI scope disclosed that apparent storm attenuation was being introduced by the radar circuits. This action may be characteristic of war surplus sets other than the APS-15. Fig. 91 is a photograph of a 584 and APQ-13 scope. The right-hand scope is the 584. This picture was presented in a Signal Corps report (3). Note the noise showing on the right-hand section of the 584 scope. Yet to the rear of the storm at upper left the scope is entirely blank as if there is a shadow. The noise originates in the crystal mixer stage so the strong signal from the storm must be causing the temporary reduction in sensitivity. This effect is not apparent in this photo of the APQ-13. I do not know if it has this same fault or not. Some 615 radar scope pictures presented in a couple of recent reports have a shadow on the far side of the storm similar to this shadow on the 584.

Fig. 92 is a picture of the Water Survey APS-15 scope on July 8, 1951 with the original circuits. The range is 100 miles. The grass level on the upper half of the scope is considerably reduced, particularly in the area immediately to the rear of the storm. The small echo 17 miles southwest (toward lower left corner) causes a very pronounced shadow and dimming of the 20- and 30-mile markers. Fig. 93 is the same scope on 30-mile range. The range markers are the third and fourth circles, the other three being signals from a 3 cm. signal generator. Both the test signal and range markers are almost invisible where rain is intervening.

Tests carried out in the laboratory indicated that the reduction in sensitivity was occurring entirely within the video circuits. The main reduction occurred in the input to the first video amplifier. Increasing all the video coupling condensers from .01 to 0.1 mfd. almost completely eliminated the sensitivity reduction. Fig. 94 is a picture of the scope taken on July 28 with the modified circuit. Isolated storms cause no reduction in the grass level. A solid line of rain from the station out to 30 miles to the southwest causes a slight

FIG. 92. —INTENSE STORM ON APS-15 SHOWING STORM SHADOW. 100-MILE RANGE.

FIG. 93. —STORM CAUSING LOSS OF RANGE MARKERS AND TEST SIGNAL. 30-MILE RANGE.

reduction in grass level. Increasing the value of the coupling condensers even more would probably reduce the effect.

ATTENUATION BY RAINFALL

During summertime thunderstorms rainfall of sufficient intensity and depth to produce considerable attenuation often exists. No actual measurements of attenuation were taken but values were calculated from the intensities and storm area as observed on isohyetal maps prepared from an eight-mile square rain gage network. Table I is a tabulation of the calculations. All the rates were obtained from one-minute amounts on the rain gage charts. This is a tabulation of some of the storms observed this summer and one example from last year. Storms on seven of the 22 days that rain fell during June and July are presented. The attenuation is computed for a two-way path through the core of heaviest rain. The attenuation rate used was .03 db/km./mm./hr. (4). Converting to miles and inches gives an attenuation rate of 1.4 db/nautical mile/inch/hr.

The values of attenuation cover a range from a minimum of 10 db on July 8 to a maximum of 56 db on July 9. The minimum rates of rainfall occurring are consistently high. The maximum rates do not necessarily give the greatest attenuation. For example, on June 26 a maximum rate of 7.2 inches per hour was recorded but the attenuation introduced by the storm was only 43 db, as compared with 56.8 db on July 9, when the maximum rate was only 3.6 inches per hour.

These high values of attenuation should be compensated for when making point or areal rainfall studies with radar. We found that often the rear of the storm was blocked out, as well as isolated storms at greater ranges. Also the core as observed on the radar may appear to be shifted toward the center of the scope from its actual location as shown on the rain gage records.

EFFECT OF DROP SIZE DISTRIBUTION

Theory indicates that when the rain drop diameter is small compared to the wave length, the power reflected is proportional to Na^6, where N is the number of drops in a given volume, and a is the radius of the drop. Various organizations working independently have arrived at similar relationships of Na^6 to rainfall intensity. Their data also

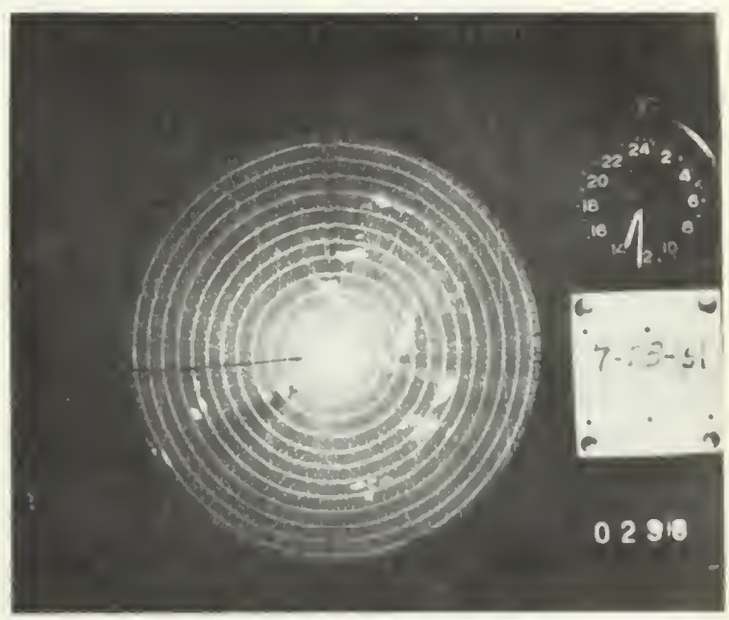


FIG. 94. —SCATTERED STORMS WITH VIDEO CIRCUITS MODIFIED TO REDUCE THE SHADOW.

Table I

Attenuation Introduced by Rainfall

Isohyetal map of one-minute amounts obtained from rain gage network. The attenuation is calculated for a radial through the core. 1.4 db per nautical mile per inch per hour, or 0.3 db per km. per mm. per hour.

Date	Time	Attenuation (two-way)	Maximum Rainfall Rate in./hr.	mm./hr.
9-20-50	1025	20 db	2.4	61
	1026	28.2	3.6	91
	1028	19.2	3.0	76
	1030	21.4	2.10	53
	1032	32.4	4.2	107
6-8-51	0052	48.6	3.90	99
6-19-51	1135	29.4	3.90	99
6-26-51		36.8	4.94	125
		43.6	7.2	184
6-27-51	2053	47.0	4.62	118
7-8-51		10.0	1.8	46
7-9-51	0012	21.2	1.8	46
	0015	56.8	3.6	91
7-22-51		15	1.2	30
	1631	16.8	2.58	65
		22.2	2.4	61
9-12-51	1717	54.6	4.8	122

indicate that there may be considerable variation in the mean drop diameter for any particular rainfall rate.

We have computed the effect the maximum and minimum mean drop diameter for a given rainfall intensity will have on the intensity computed from the return power. Two sources of data are used—Hood, and Laws and Parsons (5)(6).

Assume the radar is very accurately calibrated and the relationship $Nd^6 \pm 190\ I^{1.72}$ to be accurate. D is drop diameter and I is rainfall intensity. At 1.0 mm./hr. of rainfall $Nd^6 = 190$. However, for rainfall intensities of 1.0 mm./hr. the outer limits of Hood's data will give values as high as $Nd^6 = 800$ and as low as $Nd^6 = 110$. The radar would indicate a rainfall rate of 1.20 mm./hr. for $Nd^6 = 110$ and .43 mm./hr. for $Nd^6 = 800$. The maximum and minimum limits of drop diameters for 1 mm. rate measured by Laws and Parsons applied to the same equation result in indicated rates with a maximum of 1.45 mm./hr. and a minimum of .48 mm./hr. The relative spread in drop size at higher intensities is about the same as for the 1 mm./hr. rate, so narrower limits at higher rates would not be expected. Thus, even though the radar is very accurately calibrated, it could indicate rainfall rates from .48 to 1.45 mm./hr. for an actual rate of 1.0 mm./hr.

These limits indicate the maximum accuracy that can be expected using radar to determine rainfall intensities, unless each type of rainfall has its own particular drop size distribution and a separate radar-rainfall equation applied to each type, or that there is more uniformity in median drop size at a given rate than present data indicate.

This discussion presents a rather pessimistic viewpoint, for simple solutions are not apparent. However, these problems are specific ones that have confronted the Water Survey in using radar for quantitative rainfall measurements. These difficulties may be of minor importance when using radar for storm detection, cloud height indication, short range forecasting, etc.

ACKNOWLEDGMENT

The investigation and modification of the APS-15 video circuits was done by Mr. John C. Fatz, engineering physicist.

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2. "The Thunderstorm," Report of the Thunderstorm Project (a joint project of Air Force, Navy, National Advisory Committee for Aeronautics, Weather Bureau), U. S. Department of Commerce, Weather Bureau, Washington, D. C., 1949.
3. Brooks, H. B., "Ground Weather Test of AN/APQ-13A," Technical Memorandum No. 193-R, Evans Signal Laboratory, Belmar, N. J., April, 1946.

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4. Roberts, S. D., and King, A. P., "The Effect of Rain upon Propagation of Waves in the 1 and 3 cm. Region," Proc. of I. R. E., April, 1946.
5. Hood, A. D., "Quantitative Measurements at Three and Ten Centimeters of Radar Echo Intensities from Precipitation," Radio and Electrical Engineering Div., National Research Council of Canada, Ottawa, June 1950.
6. Laws, J. O., and Parsons, D. A., "The Relation of Raindrop Size to Intensity," Transactions, American Geophysical Union, Vol. 24, Part II.

DISCUSSION

R. WEXLER. —(Question about the first echo.)

G. FARNSWORTH. — I was concerned with the initial echo, when the first echo appears. Data presented in a report by the Thunderstorm Project stated that in most cases the maximum height of the first echo was close to the freezing level. It did build up to great heights, but the initial echo was in the vicinity of the freezing level. When it builds up, of course, the calibration would be good for a much greater range, but can we tell for sure without a range height indicator?

S. E. REYNOLDS. —Rain does not reach the ground until a number of minutes after the initial radar echo and that is what you are interested in, so the top of the radar echo can be well above the freezing level by the time rain reaches the ground.

UNKNOWN. --How much time will elapse?

S. E. REYNOLDS. —Not less than about 10 minutes.

D. D. REITER. —(Question on the video circuit.)

G. FARNSWORTH. —In the input of the first video section, following a strong positive pulse, there will be a trailing edge that will dip below the normal d. c. level in a negative direction. Our PPI scope intensity is set so that the normal d. c. level is the threshold of visibility. Therefore, when this signal dips below that, it is essentially making the scope go black. Any small signals riding down in the dip are lost. Increasing the value of the video coupling condenser increases the low frequency response. The condenser will not charge up to a high value during a strong pulse and then discharge causing this dip below. (Note. —It has been suggested that a crystal diode clamper be used to prevent the negative dip.)

MEASUREMENT OF POINT AND AREAL RAINFALL BY RADAR

BY DOUGLAS M. A. JONES AND HOMER W. HISER*

WITH DISCUSSIONS BY L. J. BATTAN, S. E. REYNOLDS, D. ATLAS, H. R. BYERS, J. C. FREEMAN, D. M. A. JONES

INTRODUCTION

The Illinois State Water Survey desires records of rainfall intensities and amounts over the entire state of Illinois. The existing sparse climatologic rain gage network does not give an accurate indication of the water that falls in a particular watershed. Fig. 95 shows the effect of varying gage density on an isohyetal pattern. It has been well established that radar will detect falling rain, giving location, areas, and vertical extents of the storm within the limitations of the equipment. It is also known that the radar echo from a storm is a function of the rainfall intensity. It is the objective of the Water Survey to establish experimentally the relationship between the received-power and rainfall intensity, and at the same time, develop a technique that will rapidly collect and record detailed information on the rainfall intensities at any point and total rainfall over the area that the radar scans.

EQUIPMENT

Radars. During the 1951 thunderstorm season the Water Survey has had available three radars: a TPL-1 10-cm. searchlight tracking radar and two APS-15A 3-cm. airborne radars. The three radars have been modified to fit the particular needs of the project (1).** The TPL-1 and one APS-15A are located on the eastern edge of the University of Illinois Airport with the radiator of the TPL-1 seven feet from the ground and the APS-15A radiator twelve feet from the ground as shown in Fig. 96. Power requirements for the radar plus the proximity of the radars to the airport runways limited the heights to which the antennae could be elevated. The second APS-15A is mounted with its antenna under a radome 70 feet from the ground on the roof of the Pfister Hybrid Corn Company factory building in El Paso, Illinois, 53 nautical miles northwest of the airport radars. It had been hoped that a clearly defined storm could be detected from both radar locations at a time when the storm was over the Water Survey dense rain gage network; but no such storm presented itself since the modification of the equipment.

Each of the three radars had an automatic system to change the receiver sensitivity in a stepwise fashion and to operate a camera so that a scope photograph is obtained on each sensitivity setting. Fig. 97 is a block diagram of the radar and recording circuit. An impulse from a microswitch on the antenna advances the stepping switch once each revolution. Taps from a series of resistors connected through the stepping switch to the i-f strip provide the bias on the sensitivity control stages. Each step the switch advances increases the bias a fixed amount, thus reducing the sensitivity. The antenna microswitch also controls a solenoid that holds the camera shutter open while the antenna makes a complete scan. Thus a picture of the PPI scope is obtained for each sensitivity setting.

Rain Gages. Thirty-four Bendix-Friez Dual Traverse Model 775-BS rain gages are used in the correlation studies. Each gage is equipped with a 12.648-inch diameter collector, and thirty-three of the gages are equipped with chart drives giving one revolution every six hours. The other gage is equipped with a twenty-four hour chart in order to orient those rains recorded by the other gages with respect to time during those intervals when the radars are not in operation. The gage at Station 31 near the center of the network has had an extra sheet metal collector 17.87 inches in diameter placed on top of its regular collector. This top doubled the rain collection area of that gage to that of the 12.684-inch diameter collector and to five times that of the standard 8-inch collector. The increased area allows a direct reading of 0.01 inch of rainfall for each vertical division of the rain gage chart.

The characteristics and performance of the Model 775-BS rain gage have been studied on a preliminary basis by R. E. Roberts (2), who reports that the instrument's charts may be trusted to report rainfall rates with an accuracy of 0.07 inch per hour if the rate is constant for five minutes or longer. The lag in time of recording at the beginning of a rainfall, for a change in rate, and at the end of a rainfall was checked. It was found that for the beginning of rainfall the lag varied from two minutes at about 0.23 inch per hour to nearly zero

*Meteorologists, Illinois State Water Survey.

**Numerals in parentheses are reference numbers in BIBLIOGRAPHY at end of this paper.

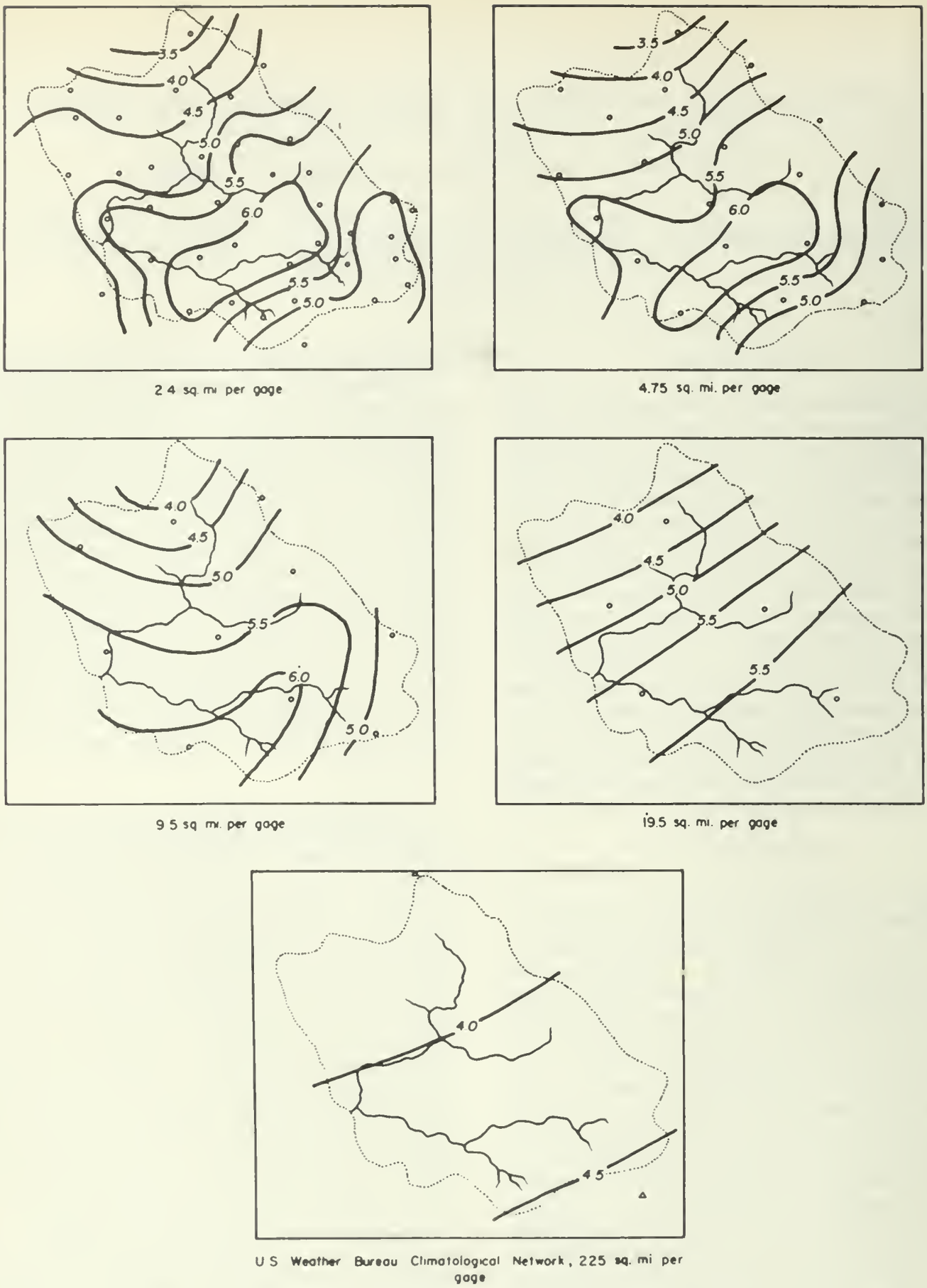


FIG. 95. — EFFECT OF GAGE DENSITY ON ISOHYETAL PATTERN. STORM OF JULY 16-17, 1950.

FIG. 96. —RADAR STATION AT UNIVERSITY OF ILLINOIS AIRPORT.

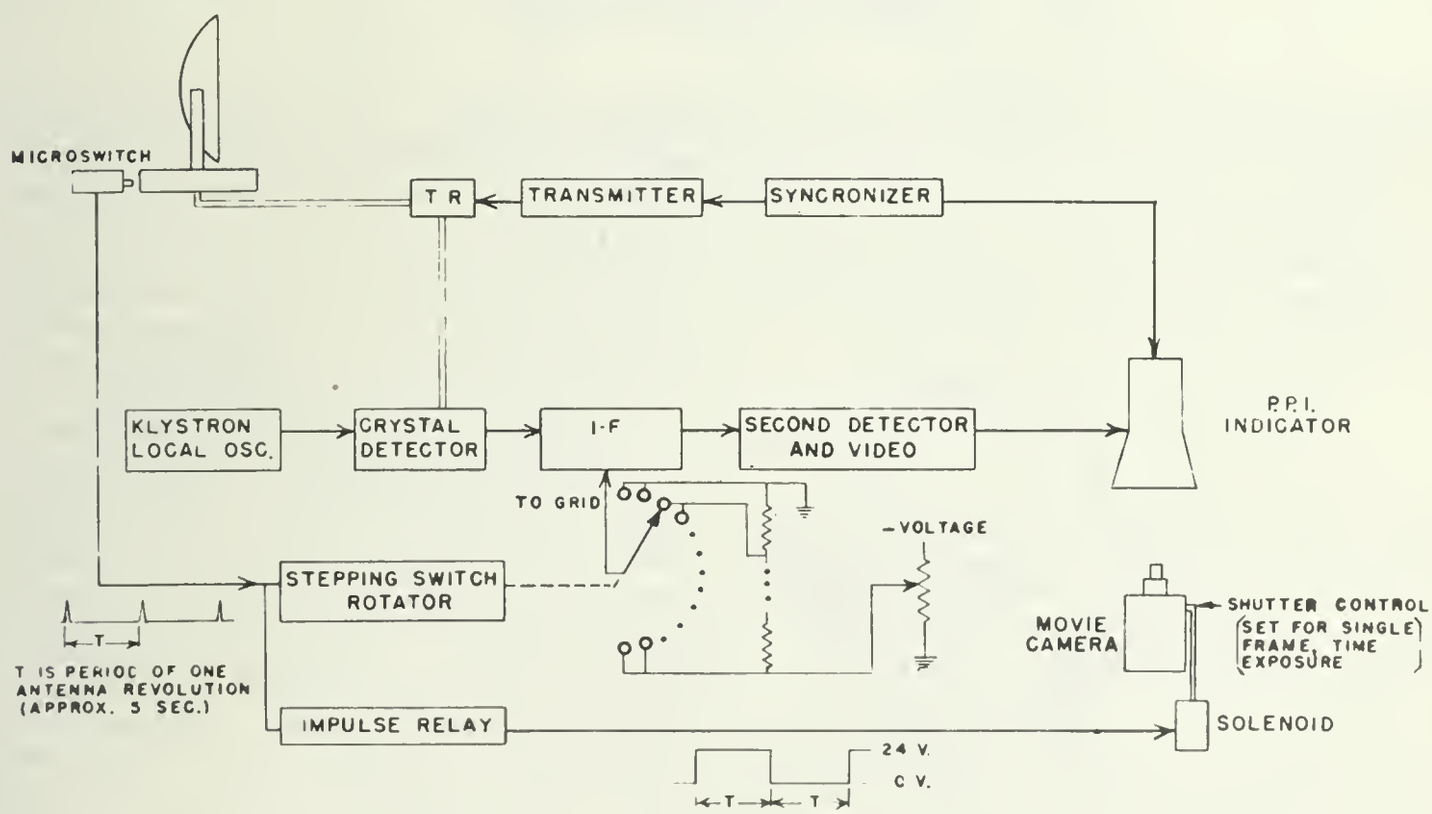
APS-15A antenna on mast and TPL-1 radar in front of well house.

for rates of about one inch per hour. Lag at the end of a rainfall was determined to vary from nearly zero at low rates to two minutes for higher rates of about one inch per hour. Lag between changes of rate during a rainfall was found to be hidden by the thickness of the trace line.

In the region of maximum sensitivity of the rain gage spring, a slight bounce of the pen arm as water drops struck the bottom of the rain gage bucket was noticed. This slight movement caused a pronounced broadening of the trace line. In many cases, this change in width of the trace line indicated the beginning of a light rain not intense enough to record as a rising trace.

Rain Gage Network. The dense rain gage network of thirty-four gages is located on the Goose Creek watershed between 15 and 22 nautical miles west-northwest of the airport radar station (Fig. 98). The rain gages are laid out on radials 3 1/2° apart with their origin at the airport radar station. A stream-gaging station has been installed on Goose Creek in order to utilize the rain gage records in runoff studies.

1950 Observational Program. A micro-network



RADAR WITH AUTOMATIC RECEIVER SENSITIVITY AND CAMERA CONTROL

STATE WATER SURVEY
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FIG. 97.

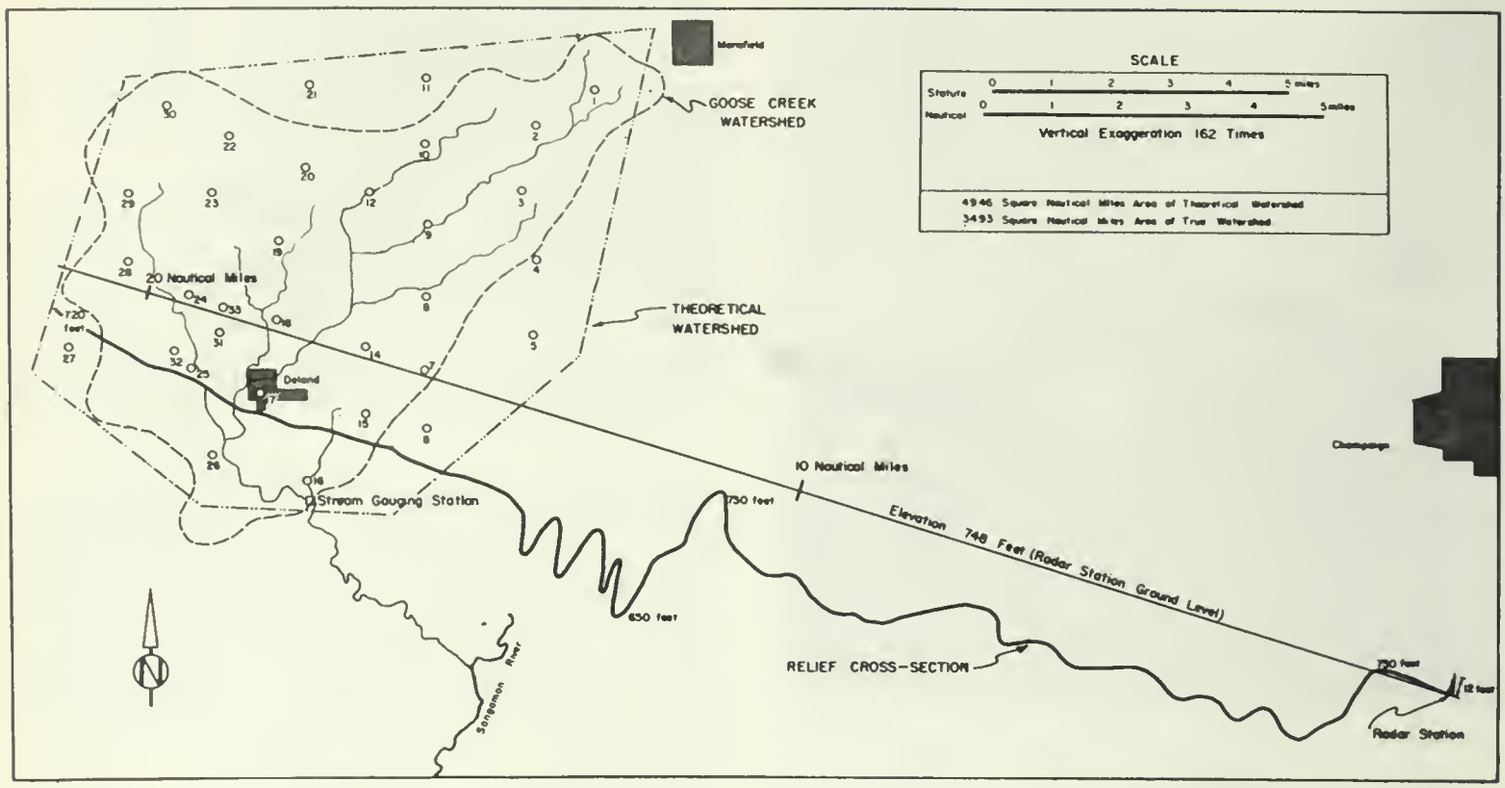


FIG. 98. —AIRPORT RADAR STATION AND GOOSE CREEK RAIN GAGE NETWORK.

of 30 of the rain gages described above was placed near Washington, Illinois, on the Farm Creek watershed during the 1950 thunderstorm season. The U. S. Weather Bureau cooperative observer's rain gage in Washington made a total of 31 rain gages available during the observational program. Washington, Illinois is situated 17 nautical miles west of the El Paso 3-cm. radar, which was the only radar used for analysis during 1950.

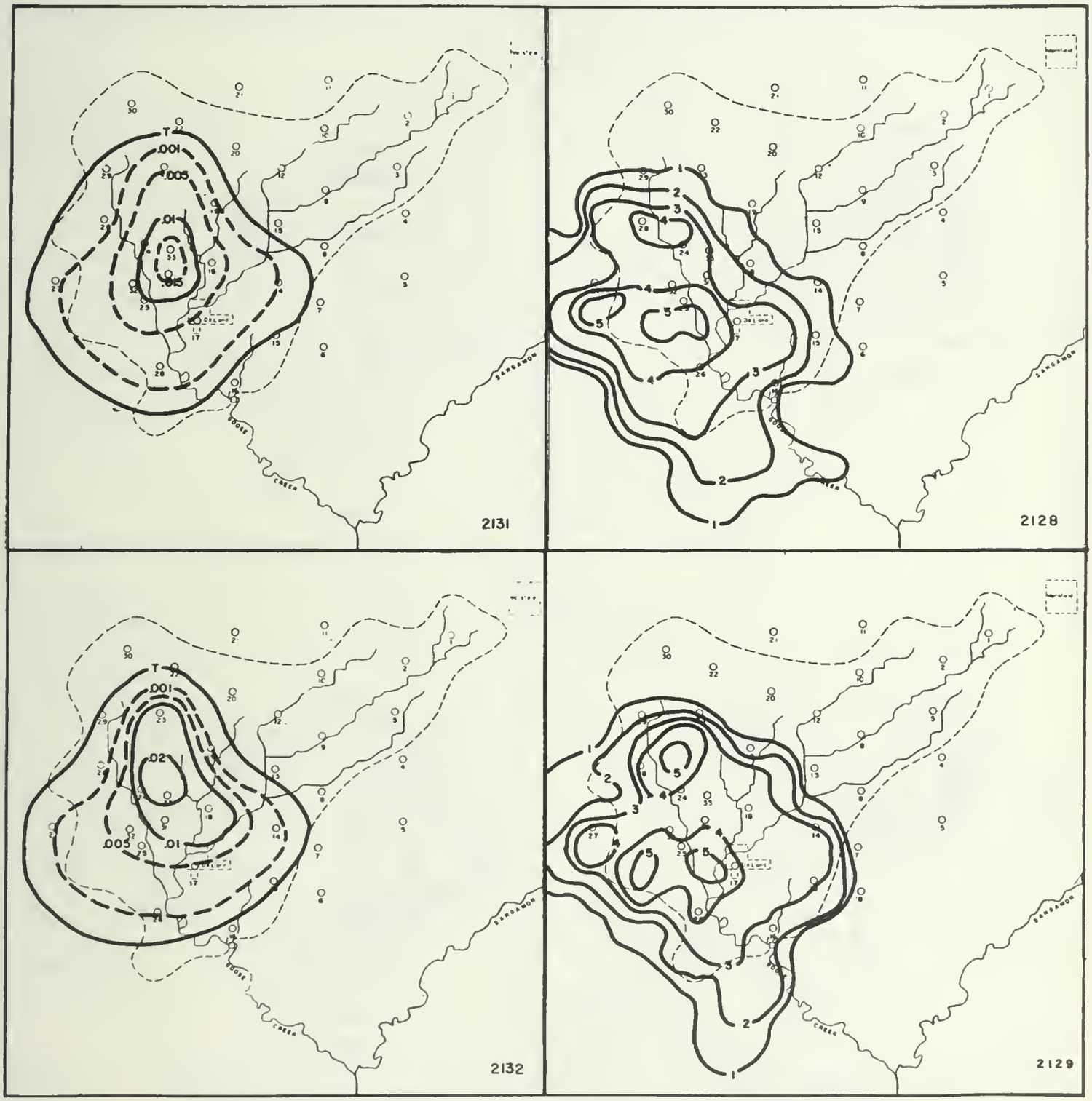
DISCUSSION

APS-15 transmitter power output and receiver sensitivity measurements were made with a war surplus test set, the TS-263. The thermistor bridge circuit and calibrated attenuator permitted relative reading in power output to ±1.5 decibels. Absolute accuracy is estimated at ±3 db.

It would be desirable in calibrating the return from radar in rain measurement to measure the instantaneous rate of rainfall. An attempt was made to design a "rate-reader" which would operate on the principle that a tangent to the rainfall trace is an indication of the rainfall rate at that point; but it was found that an easier (although less precise) method was to read one-minute rainfall cumulative totals and assume these to be rates. This introduces a time error as well as a statistical error in that the one-minute total rainfall when taken as a rate, shifts the time of the observation from the beginning or ending of the particular minute to the middle of that time interval. However, the finite length of time required for the radar to complete a series of gain steps, of the order of one minute, tends to make the radar record extend over the same time interval as that covered by the rain gage record. For precision it would be desirable to have a network of instantaneous rainfall rate recorders and a radar modified to present a full series of gain contours in one sweep of the antenna.

Perhaps the largest source of error found in the observational program has been the lack of synchronism of the times of the various recording instruments. Generally, the time errors are unknown variables. This has been due to a number of causes, among which are: (1) radar and rain gage operator error, (2) non-uniform clock speeds on the rain gages and the radars, and (3) errors introduced by the design of the rain gage chart. This last named error is introduced by the space left between lines of the time scale on each end of the rain gage chart when the chart is wrapped around the six-hour clock cylinder. The error becomes apparent when the recorder pen passes over this joint and is of the order of 0.25 to 2 minutes. Correction for such an error becomes subjective in that four such time lapses occur in each twenty-four-hour period and for accurate calculation of the error the time lapse should be read at least to the nearest quarter minute. The magnitude of this error has been forcefully

illustrated a number of times by comparison of the rainfall traces of two rain gages set ten feet apart with equal exposure. When the significant points on the two curves were checked against each other as to time, it was found that all methods of correcting clock errors would not reconcile the two records to a common time scale even though the time lapses between significant breaks in the traces were found to be closely comparable. The only explanation offered is that the error lies in the correction for the break between the ends of the time records of the individual charts.

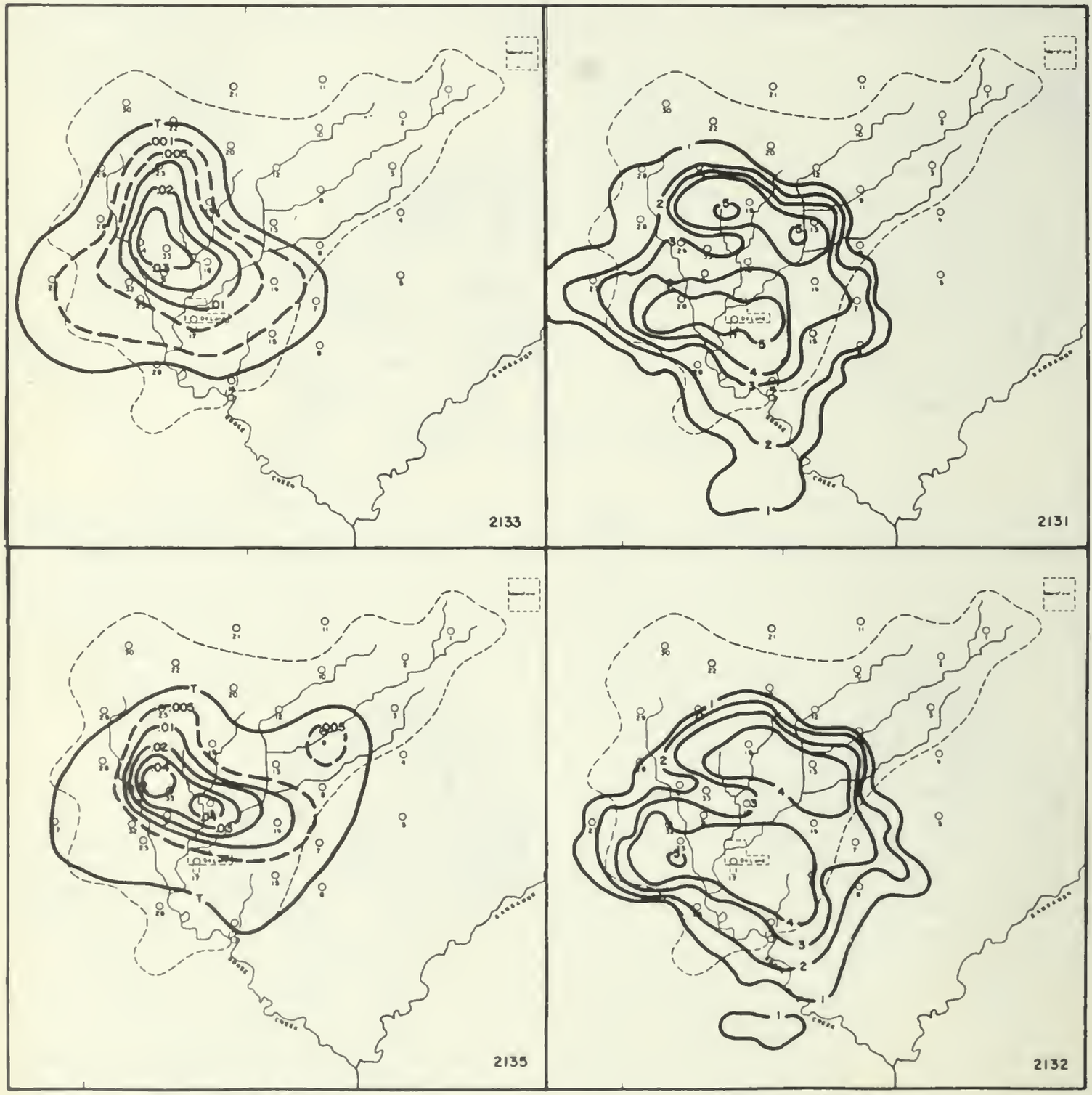


ONE MINUTE MAPS, JULY 22, 1951

FIG. 99.

ANALYSIS

Radar. While rain is over the Goose Creek rain gage network, photographs are taken of the PPI scope on every second scan of the antenna. Use of the stepping switch at the time photographs are being taken results in a series of pictures in which the echo area is reduced in a stepwise fashion with the smaller areas representing the more intense portions of the shower. The number of steps required to clear the entire echo pattern from the scope is chosen by the radar operator. About seven



ISOHYETAL PATTERN

ISOECHO PATTERN

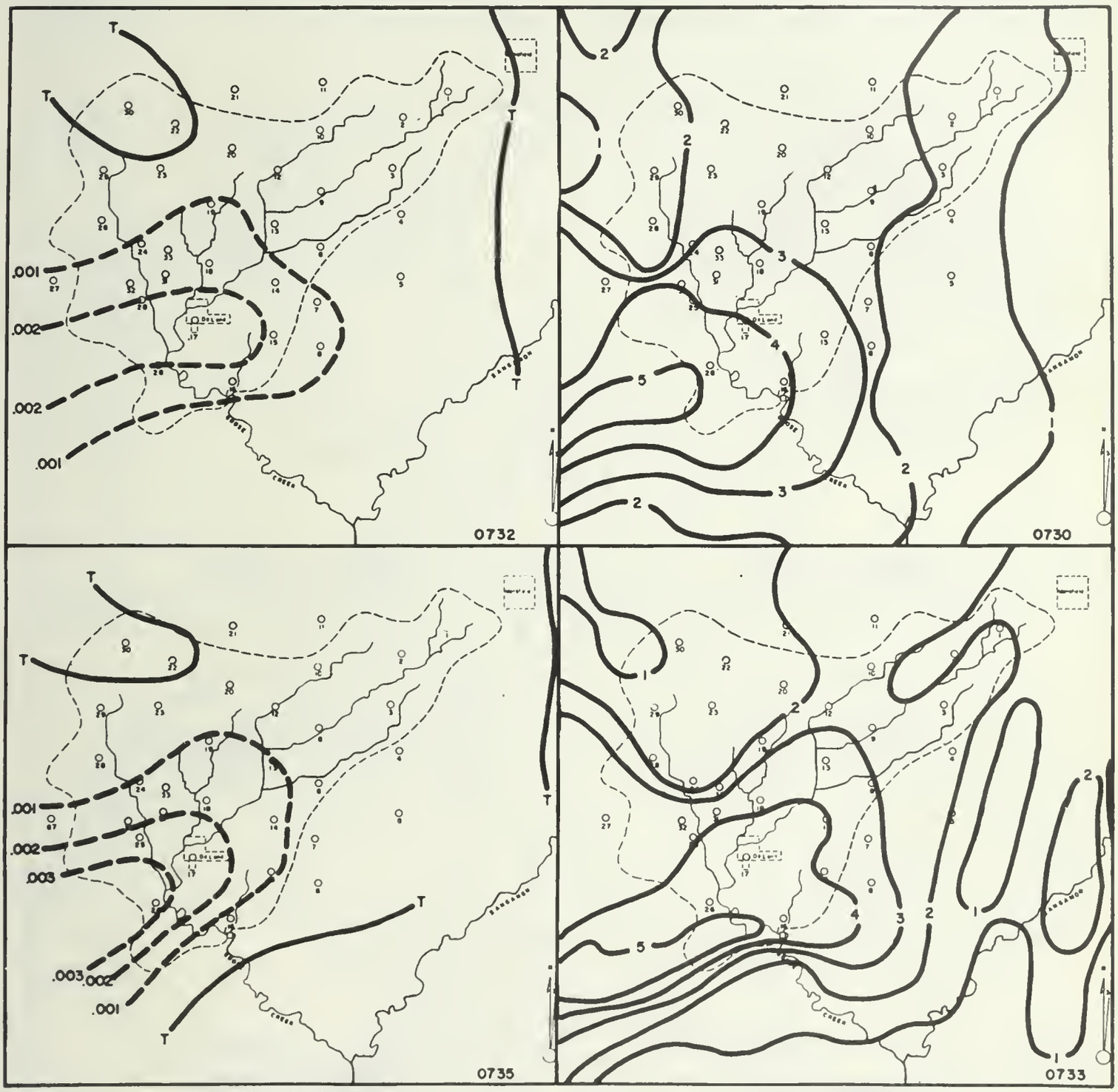
ONE MINUTE MAPS, JULY 22, 1951

FIG. 100.

steps (equivalent to a total gain reduction of about 30 db) are required for most showers with one-minute amounts over .05 inch. Time consumed in photographing these seven steps is about 1 1/2 minutes.

These photographs of the radar echoes were projected on a base map of the rain gage network, and the outline of the echo was drawn for each step. In most cases these could be superimposed one on the other to give a map of the shower as shown by the stepping switch (Fig. 99 and 100).

Rain Gages. The rain gage charts from the 34 recording rain gages on the Goose Creek network were each enlarged eight diameters by projection



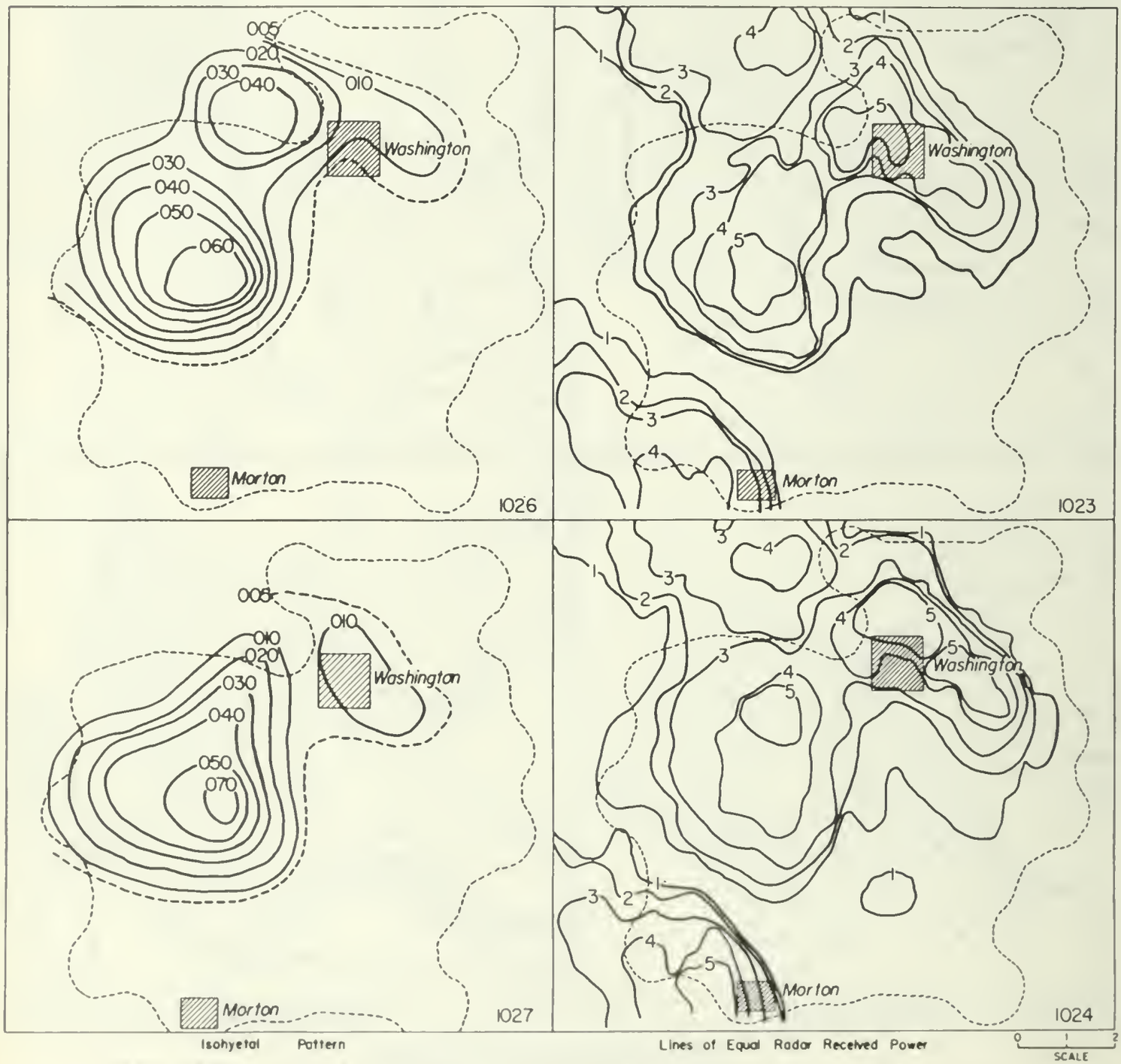
ONE MINUTE MAP, JULY 22, 1951

FIG. 101.

so that one-minute rainfall values could be read from them. These readings could be made quite accurately to 0.005 of an inch and smaller values could be approximated satisfactorily.

These one-minute rainfall values were plotted on base maps of the Goose Creek network and one-minute isohyetal maps were drawn. These isohyetal patterns were correlated with the radar isoecho maps as described later.

The question will no doubt arise as to why one should use such short intervals as one minute for the radar and rainfall correlation maps. We have found a great deviation in both the radar and rainfall patterns from minute to minute. After numerous trials with five-minute maps and other intervals, it was finally decided that the best similarity of patterns was obtained using the shortest practicable time period—one minute. For light rains of the warm front type which show only slight intensity variations with time, such as those in Fig. 101,



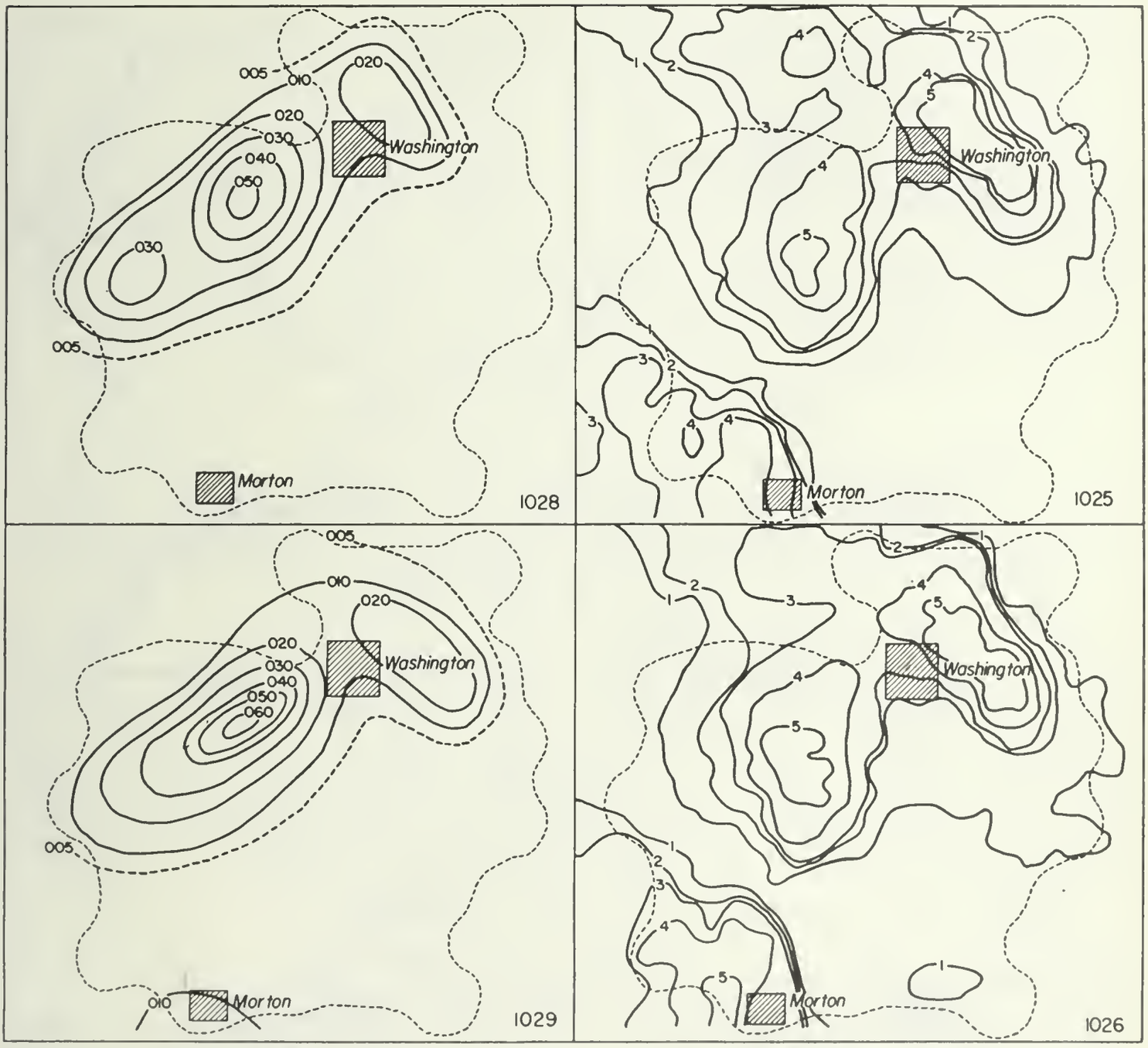
STORM RAINFALL PATTERN AS SEEN BY RADAR AND BY RAINGAGES Date 9-20-50

FIG. 102.

longer intervals could no doubt be used with a considerable degree of satisfaction. Most of the data are from thundershowers that show many short period intensity fluctuations on the rain gage charts and radar photos as well as high rates of rainfall, as illustrated by Fig. 102, 103, 104, and 105, which were taken from the 1950 data.

Radial Method (Point). We have used two methods of correlating the radar isoecho maps with the one-minute isohyetal maps—the radial and the areal methods.

With the 1950 data (3) it was necessary to concentrate on leading edge correlations due to a scarcity of thunderstorm cells within our rain gage network and a considerable amount of apparent attenuation, a large portion of which was later revealed to be introduced by the radar circuits (1). Each gain step on the radar isoecho map was matched with a certain isohyet on the rainfall map. To do



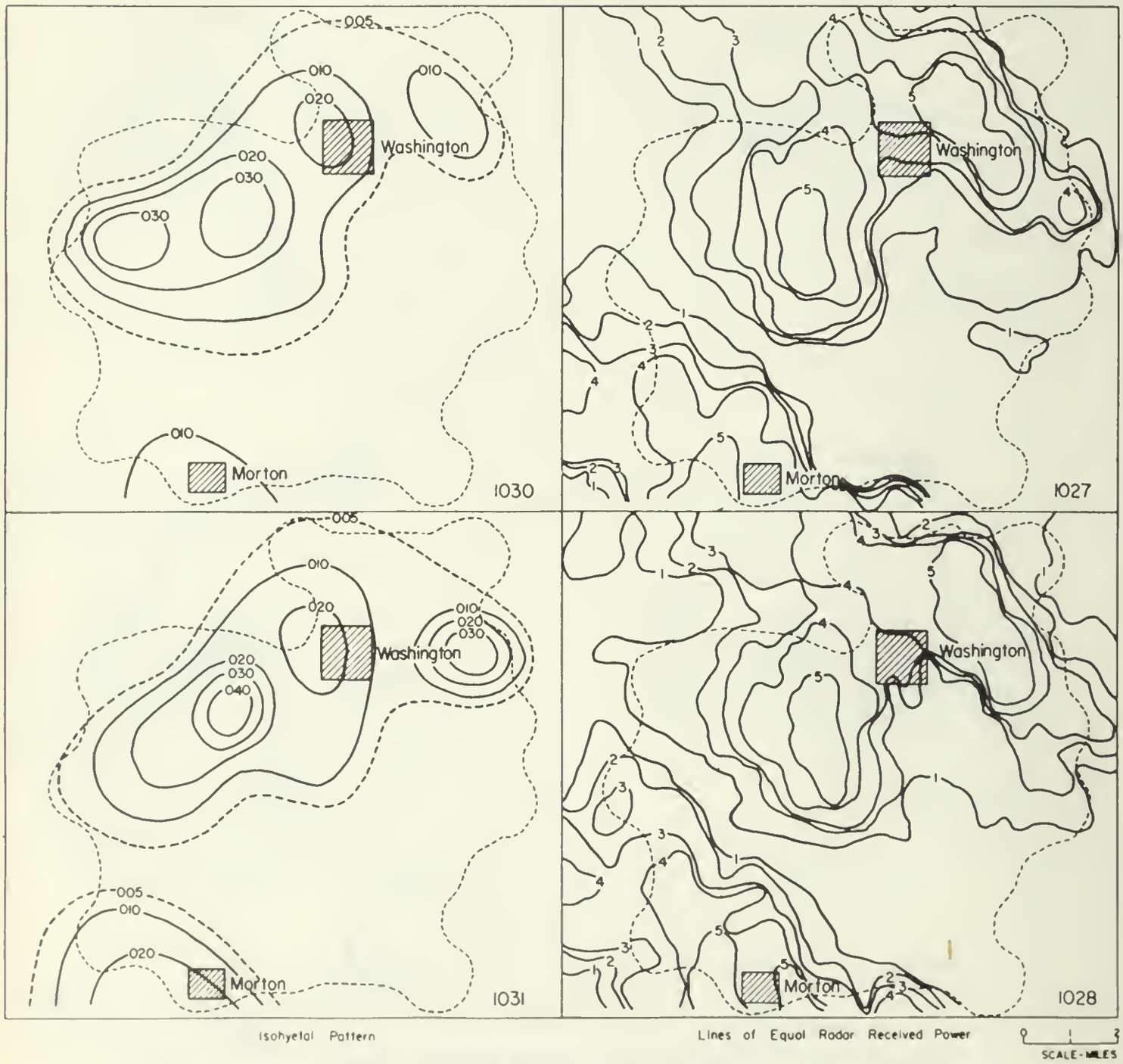
STORM RAINFALL PATTERN AS SEEN BY RADAR AND RAINGAGES DATE: 9-20-50

FIG. 103.

this a reasonable lag time of 1 to 3 minutes was allowed for the rain to fall from the radar level to the ground and used a drift of 0 to 1 1/4 miles, both of the above depending on the particular situation, wind, probable rate of fall, etc.

A convenient method of analysis for this leading edge method proved to be the use of radials from the radar station. Profiles of the radar echo in terms of range and power-received were compared with similar ones of range versus rainfall rates in inches per hour. It was by this method of comparison that the curves relating range, rainfall and returned power were constructed for the preliminary report completed in the spring of 1951 (3).

Areal Method. While analysis of the 1950 data was proceeding, it was felt that perhaps an areal basis of correlation would be more satisfactory if more cells could be observed within our rain gage network and some of the attenuation introduced in the radar set could be reduced. As mentioned earlier, the radar was modified to reduce the internal



STORM RAINFALL PATTERN AS SEEN BY RADAR AND RAINGAGES DATE 9-20-50

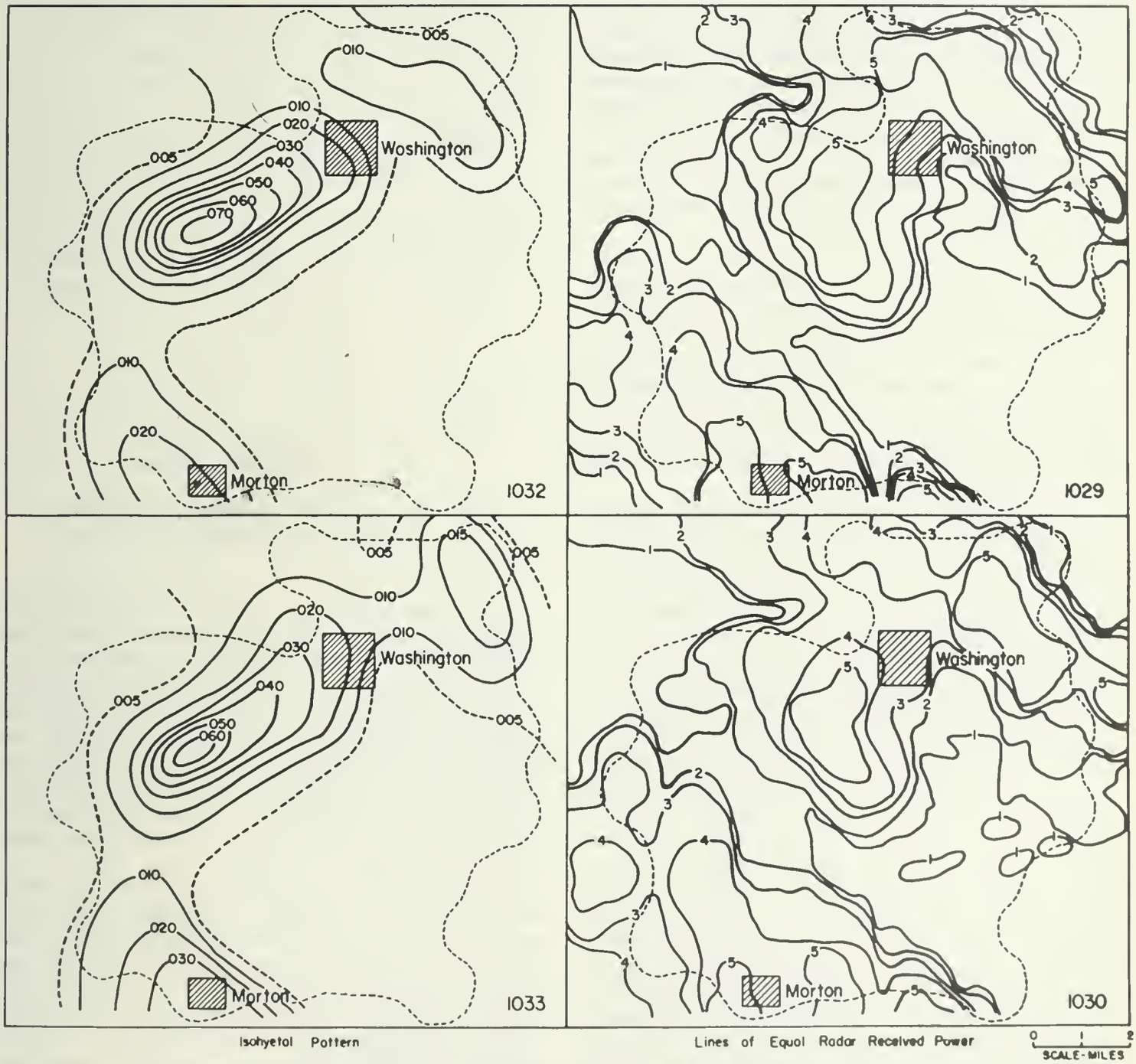
FIG. 104.

circuit attenuation and permit the weaker signals from the rear of the showers to show up on the scope, thus giving a more representative picture of the area of rain. During 1951 there were also a few cells passing near the center of the rain gage network.

The areal method described is somewhat similar to that suggested by Byers and collaborators (4), in which horizontal cross-sectional areas of different echo intensities are considered to be representative of different volumes of rainfall.

In assembling the data, values were used only for cores of rainfall located well within the boundaries of a "theoretical" watershed. This theoretical watershed is outlined by the dashed polygon in Fig. 98.

For each of the one-minute intervals considered suitable for comparison of radar return-signal strength and rainfall areas of perceptible return recorded on the scope photographs for the radar gain steps that lay within the boundaries of the theoretical watershed were planimetered, as were the



STORM RAINFALL PATTERN AS SEEN BY RADAR AND RAINGAGES DATE 9-20-50

FIG. 105.

area of isohyets within the watershed, beginning with the trace line which indicated the outer boundary of the rain. The isohyetal areas were plotted against the value of rainfall that was equalled or exceeded to produce an "area-depth" curve for each of the one-minute intervals examined. Greater reliance was placed on values greater than 0.001 inch per minute in drawing these area-depth curves. The areas of perceptible response on the radar scope were tabulated against the corresponding "step" settings.

To obtain the rainfall value corresponding to each of these radar values, the area recorded for each was used to enter the corresponding "area-depth" diagram, from which the appropriate rainfall value was interpolated. An area-depth curve one to two minutes later than the radar time was used to allow for time of fall of the rain from the level where the radar beam 'saw' it to the ground where the rain gage network recorded it. The interpolated rainfall values were averaged for each step and these averages were plotted against the gain steps for various rainstorms, producing the curves shown in Fig. 106.

Results. Plotted on Fig. 106 are five curves correlating the radar received power to the rainfall rate during certain rainstorms of 1951 by the use of the areal method of analysis. Also included on Fig. 106 are curves, derived in 1950 by the radial method by this Survey, the average computed power return to be expected during 1951 using the radar equation of Marshall, Langille, and Palmer (6), and a curve developed by Atlas (5) in 1948.

Curves I, II, and III were all derived from one storm day, July 22, 1951; but it will be noted that these three curves include for a given rainfall rate the greatest power received the least power received, and the approximate average of the logarithms of the powers received! These curves indicate that one or both of two processes are taking place. One process is the possibility that the transmitted power was decreasing or the sensitivity of the radar was changing to a less sensitive value during the day; and the other possible process would be a significant change in the mean drop diameter or reflectivity for a given rain intensity. It seems possible that the radar parameters changed during the day. No power measurements were made on July 22. A power measurement was made on July 17 when the peak transmitted power was found to be 27 kilowatts and the receiver threshold of visibility 85 db below 1 milliwatt. Another power measurement was July 24 when the peak transmitted power was found to be 21.5 kw. and the receiver threshold of visibility 93 dbm. It will be noted that Curves I and II change in slope at approximately 1.5×10^{-13} miles^2 and it is conceivable that Curve III would have changed slope at the same value had the power received reached that value.

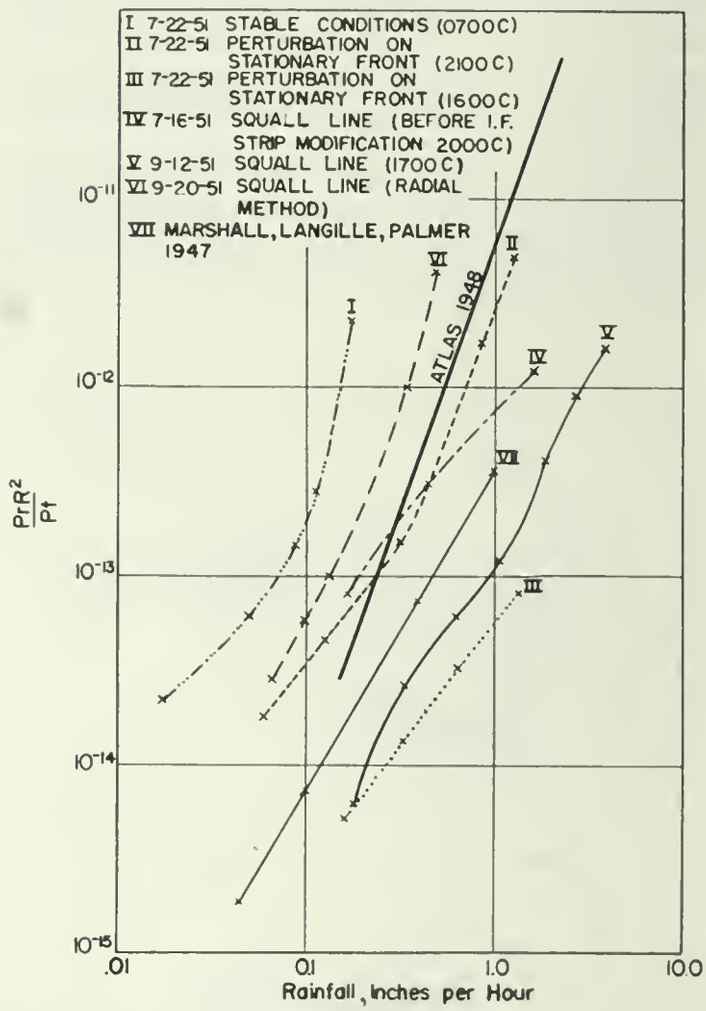


FIG. 106.—POWER-RANGE FACTOR VS. RAINFALL INTENSITY.

The data determining Curve IV were collected before the modification of the radar i-f strip and included rainfall rates as high as 1.64 inches per hour, which rainfall rate would cause attenuation of the reflected signal. It would appear that the attenuation from these two combined sources has caused the radar to indicate a decreased logarithm of the power received for a given higher rate of rainfall. It is thought that the apparent attenuation formerly present within the i-f strip was the major factor causing this decrease.

Curve V has a shape different from all other curves derived, which shape is thought to be due to the attenuation resulting from very heavy rain within a small area and rain of one inch per hour covering a very large area, causing the light rain on the side of the storm away from the radar to be completely obscured and thus have a much smaller radar echo area than is actually the case. It will be noted that this curve follows the slope of the calculated curve quite closely and lies approximately 5 decibels below it. The range of rainfall 0.18 to 0.36 inch per

hour on this curve tends to follow a slope similar to that of Atlas, but changes from 0.36 inch per hour to a slope less than that of the calculated slope. No explanation is offered for this phenomenon. It must be remembered that these curves are plotted from points derived by averaging all values of power received for a given rainfall rate and drawing a smooth curve through those points by estimation, while Atlas' curve is a regression curve for a straight line. Another possibility in explaining the peculiar shape of Curve V might be the Rayleigh scattering effect at 3.2 cm. wave length as suggested by Wexler (7). This effect may be active in the change in slope noted in all the other curves since it will be noted that the breaks in slope occur in the same range of rainfall rates indicating that this is the range of points at which certain drop-sizes become most effective in reflecting the radar beam. This does not hold too well for Curves II, III, and VI. However, it will be noted that the latter named curves all occur under a certain synoptic weather condition, a perturbation on a stationary front, suggesting that there is a significant shift in drop-size distribution from one synoptic situation to another.

Curve VI has been derived from the data of September 20, 1950, and has been analyzed by the radial method when it was assumed that rainfall attenuation at 3.2 cm. wave length has little effect, since the only data used have come from the leading edge of the storm to the core of heaviest rainfall rate. Since the original analysis of this 1950 data, it has been determined that such an assumption is not valid and that the attenuation from the leading edge to the core of heaviest rain may be as high as 20 db for two-way transmission through a storm 12 miles in diameter with a center core of rainfall intensity of 4 inches per hour. In Curve VI attenuation due to rainfall has been masked by the increased power return caused by the deviation of the reflectivity, as predicted by the Rayleigh scattering law, as the diameter of the rain drops approaches the wave length of the radar.

ACKNOWLEDGMENT

The authors are indebted to Mr. H. E. Hudson, Jr., Head, Engineering Subdivision, for guidance in writing this paper.

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DISCUSSION

L. J. BATTAN. —Since this area idea is directly applicable to hydrology and since the area was only a basin, I wonder if any efforts were made to tie in this information with that of the stream gage.

D. M. A. JONES. —A stream gaging station is placed on Goose Creek, and the radar and rain gage data will be correlated with it; but, as yet, this has not been done.

S. E. REYNOLDS. —It seems to me that since the radar beam width is around 6,000 feet at 20 miles, it in effect integrates the rainfall over a number of minutes and, therefore, using a resolution of one minute on the rain gage and then comparing that to the intensity of the radar return, will introduce some discrepancy.

D. M. A. JONES. —That is no doubt true. We wondered how good our correlation can be for that reason. We have a three-degree beam width now, and we will have a one-and-one-half-degree beam width later to reduce the area. As I mentioned in the paper, we found that the five-minute intervals did not work at all; we found no correlation whatsoever.

S. E. REYNOLDS. —That is what is hard for me to believe. It seems the longer the period, the better the correlation should be.

D. M. A. JONES. —It is because the rate changes too fast.

S. E. REYNOLDS. —But how can the radar see this?

D. M. A. JONES. —I think that part of the solution may be in the fact that we try for a zero degree tilt on our antenna. We may be mistaken in this, but we thought we would get the best correlation by trying to see the rain at the ground. Of course, we still have some of the beam in the air.

S. E. REYNOLDS. —This means that you are only covering about 3,000 feet, then, in 20 miles.

D. ATLAS. —It seems to me that there is quite a group of us here who have worked in correlating and have paid attention to rainfall a number of years. My personal idea is that, in my work and in that of the Illinois State Water Survey, it has been shown that we have just about reached the limit of accuracy and that we should recognize that the relationship between drop measurement has certain dissimilarities.

H. R. BYERS. —When it comes to the practical determination of rainfall, we are concerned with rain that occurs over a period of time; these measurements that we have been talking about have been correlations within a minute, and instantaneous and so forth, and I think that the data of the Water Survey and other data will bear out the fact that the time factor is extremely important in determining the total amount of rain that falls over a given spot. In the case of these discrepancies time is a great healer, or a great smoother of the differences, and I think that over a period of a storm, particularly if it is staying over one position for any length of time, you will find that the relationship is good because the time factor enters into it, and it is a very important point.

J. C. FREEMAN.*—I wanted to say one word as an observer and as a meteorologist. I do not want you radar men to forget what a wonderful thing it is that you can sit in a little shed somewhere and get an idea of the relative intensity of the rain over a large area. So let us get something out of it so everybody can use it.

*Meteorologist, Cook Research Laboratory, Chicago, Illinois.

REDUCTION OF FLUCTUATIONS IN ECHOES FROM RANDOMLY DISTRIBUTED SCATTERERS*

BY J. S. MARSHALL[1] AND WALTER HITSCHFELD[2]

WITH DISCUSSION BY W. HITSCHFELD, M. H. LIGDA, J. S. MARSHALL

The subject matter of this paper is an adaptation of fairly well-known principles of the theory of fluctuations to radar weather, leading to some practical conclusions of interest in this field.

Radar returns from precipitation or cloud are subject to serious fluctuations. Quantitative interpretation of radar displays—A-scopes or brightness-modulated presentations—are rendered very difficult by the grassy, noise-like nature of the signal. These fluctuations are amenable to some degree of mathematical description by means of probability distributions.

Let us first consider a pulse of length h traveling out from the radar. At any one instant, the signal received comes from scatterers contained in a region in space—the contributing region—which is a section of the beam of length h/2. The signal can be represented by a phasor both in magnitude and direction. This phasor is the resultant of component phasors each originating from a series of slices of the contributing region which are all scattering in the same phase.

If one considers an arbitrary volume V containing many uniform scatterers, whose mean density is $\bar{n}$ and each contributing an amplitude a, all in the same phase, then the amplitude of the signal scattered from this volume V at any instant, consists of a fixed mean value $V\bar{n}a$ plus the fluctuation whose magnitude has a Gaussian probability distribution, of mean zero and of mean square value $V\bar{n}a^2$.

If one resolves these amplitudes into rectangular components, and sums over all phases, one obtains Gaussian distributions of mean zero and mean square value $1/2\ V_c n a^2$, where V_c is the volume of the contributing region. It is noteworthy that $\bar{n}$ and a enter this result only in the combination $\bar{n}a^2$. Consequently, if the scatterers are not uniform (as indeed they rarely are), the result is still valid if for $\bar{n}a^2$ the effective average $\overline{\Sigma}a^2$ (for unit volume) is substituted. It follows also that

$$\overline{A^2} = \overline{X^2} + \overline{Y^2} = V_c\ \overline{\Sigma}a^2,$$

which expresses the well-known result that the average value of the total intensity scattered equals the average value of the sum of the intensities contributed by the scatterers, the squares of the amplitudes being proportional to the intensities.

It is also easy to derive the probability distributions of the signal amplitude A, its square A^2 (proportional to the signal intensity), and log A^2 (the signal intensity level).

The results, and some properties, such as mean and most probable values, are summarized in the table, Fig. 107. We note that all these distributions and all their properties are expressible in terms of one parameter, $\overline{A^2}$, a quantity proportional to the mean value of the received intensity; this quantity in turn is proportional to $Z = \overline{\Sigma} D^6$, the sum of the sixth powers of the particle diameters, when Rayleigh scattering is applicable. This suggests in fact that this is the ONLY quantity the radar is capable of measuring. Directly, the radar can give us no more than contour maps of Z. Meteorological quantities—such as particle density or size, or mass of water in unit volume or intensity of precipitation—can be deduced only if other physical information is available, as was discussed last night, particularly by Mrs. Austin.

The distributions of A^2, A and log A^2 are seen on the left of Fig. 108. They convey the extent and seriousness of the fluctuations; they emphasize the necessity of greatly reducing these fluctuations, which can only be done when many independent signal values are studied at the same time.

One way of doing this is to average the independent signals. The probability distribution of averages of k independent intensity measurements has been worked out, and is shown in Fig. 109 for several values of k. It will be noticed how radically the exponential distribution law (for k = 1) changes as k increases. The standard deviation about the true mean $\overline{A^2}$ decreases inversely with the square

*The research reported in this paper has been sponsored by the Geophysics Research Division of the Air Force Cambridge Research Center, under Contract AF 19(122)-217. The paper was presented by Walter Hitschfeld.

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2.6 Summary of the Above Statistics

(a) The distributions:

$$P(X) = \frac{1}{(\pi \overline{A^2})^{\frac{1}{2}}} \, e^{-X^2/\overline{A^2}} \, , \qquad \text{similarly for } Y.$$

$$P(A) = \frac{2A}{\overline{A^2}} \, e^{-A^2/\overline{A^2}} \; ; \qquad P(A^2) = \frac{1}{\overline{A^2}} \, e^{-A^2/\overline{A^2}}$$

$$P(\log A^2) = \frac{\exp\left[\log A^2/M - (e^{-\log A^2/M})/\overline{A^2}\right]}{M \, \overline{A^2}} \qquad M = \log e$$

Here $\overline{A^2} = (Bh/4) \sum e^2$ is proportional to the mean value of the intensity received.

(b) Mean and most probable values:

$$\overline{X} = 0 \; ; \qquad X_p = 0 \; ; \qquad \overline{X^2} = \tfrac{1}{2} \overline{A^2} \; ; \quad \text{similarly for } Y.$$

$$\overline{A} = \tfrac{1}{2}\sqrt{\pi \overline{A^2}} = 0.88623 \sqrt{\overline{A^2}} \; ; \qquad (\overline{A})^2 = 0.78540 \, \overline{A^2}$$

$$A_p = \sqrt{\overline{A^2}/2} = 0.70711 \sqrt{\overline{A^2}} \; ; \qquad (A_p)^2 = 0.50000 \, \overline{A^2}$$

$$\log^{-1} (\overline{\log A^2}) = \overline{A^2}/e^{\gamma} = 0.56146 \, \overline{A^2}$$

$$\log^{-1} (\overline{\log A}) = (\overline{A^2}/e^{\gamma})^{\frac{1}{2}} = 0.74929 \sqrt{\overline{A^2}}$$

$$\log^{-1} (\log A^2)_p = \overline{A^2} \; ; \quad \log^{-1} (\log A)_p = \sqrt{\overline{A^2}}$$

(c) Median and quartile values:

(The suffix m denotes median, the suffices q_1 and q_2, the lower and upper quartiles respectively.)

$$\left.\begin{aligned} X_m &= 0 \\ X_{q_1} &= -0.47729 \sqrt{\overline{A^2}} \\ X_{q_2} &= 0.47729 \sqrt{\overline{A^2}} \end{aligned}\right\} \text{ similarly for } Y.$$

The median and quartile and, indeed, any percentile points of the various distributions derived from A all occur at corresponding places:

the lower quartile where $(A^2)_{q_1} = 0.238 \, \overline{A^2}$

the median quartile where $(A^2)_m = 0.692 \, \overline{A^2}$

the upper quartile where $(A^2)_{q_2} = 1.386 \, \overline{A^2}$.

FIG. 107. —DISTRIBUTION LAWS OF SIGNAL AMPLITUDE, INTENSITY AND INTENSITY LEVEL, AND SOME OF THEIR PROPERTIES.

root of k, and the peak is to be found at $\overline{A^2}(1 - \frac{1}{k})$ so that it approaches the true value as k increases.

A more useful way of exhibiting the results is shown in Fig. 110, where we have plotted the limits below which 2.5, 10, 25, 75, 90 and 97.5% of the averages of k independent values may be expected to lie. The graphs constitute essentially the 5, 20 and 50% confidence limits, plotted as a function of k. It will be seen, for instance, that 80% of the averages of 50 independent measurements of A^2 fall into the interval $\overline{A^2} \pm 0.2 \, \overline{A^2}$.

A purely analytical investigation of the same problem for signal amplitude A and intensity level $\log A^2$ proved too complex. We therefore undertook a numerical study (by studying statistically a sample of 1,000 independent values of A and $\log A^2$ computed from tables of randomly-sequenced Gaussian deviates) and found that the confidence limits

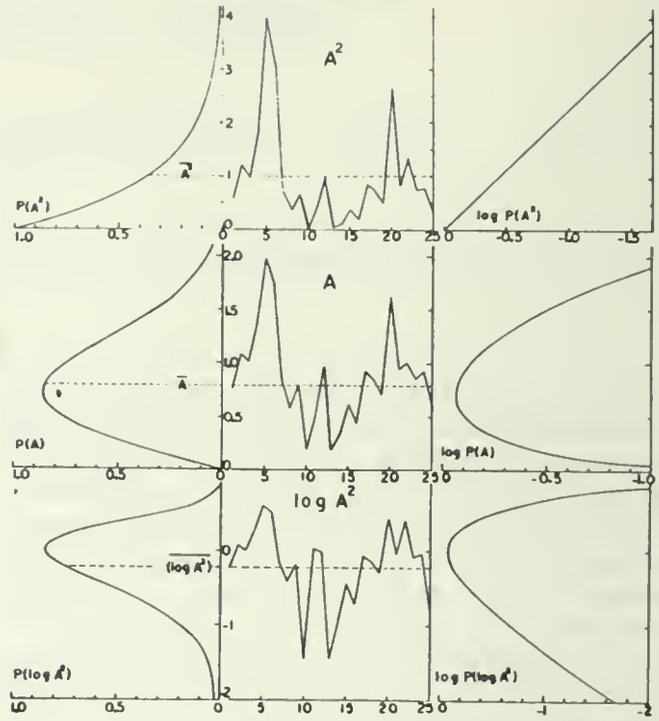

FIG. 108.

Top row (left to right): probability distribution of signal intensity (A^2), typical traces of independent A^2-values, and distribution of luminance on "A^2-scope" resulting from superposition of many independent A^2-traces. Center row, same for signal amplitude, A; bottom row for signal intensity level, $\log A^2$.

within which these functions fluctuate about their own means are about the same as those for A^2, when they are expressed in comparable terms. Only when k is small, do A and $\log A^2$ presentations seem to be slightly superior to those of A^2. We deduced from this that we should not limit ourselves to either linear or square-law presentation. Logarithmic amplifiers and presentations are as good in this context, and superior in some others, as will be pointed out below.

If one is concerned with the evaluation of A^2 from single or average readings, a more pertinent way of looking at the problem is to evaluate the probability distribution of the true average in terms of an average of k independent signal values. The resulting distribution is rather like the one for A^2—particularly for the higher values of k—but is in general somewhat broader.

Another way of utilizing several independent signals is by classifying them according to whether they fall above or below a given threshold. The fraction p/k of the signals above the threshold can be used to compute a value of the signal average. In Fig. 111 is shown the precision attainable in this way in terms of k and the long-term average of the ratio p/k. It would appear, for instance, that when the threshold is chosen to correspond to $\overline{A^2}$

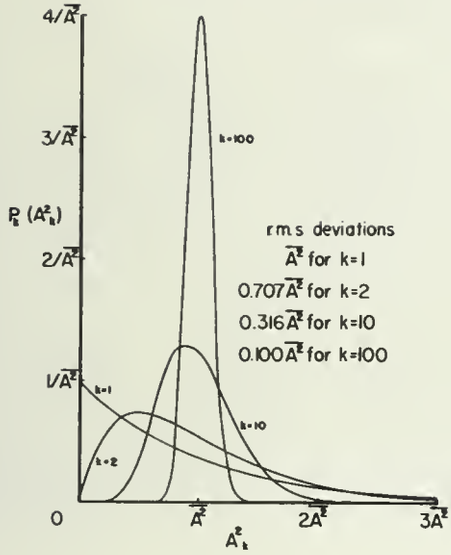

FIG. 109.—PROBABILITY DISTRIBUTION OF J_k, THE AVERAGE OF k INDEPENDENT VALUES OF A^2.

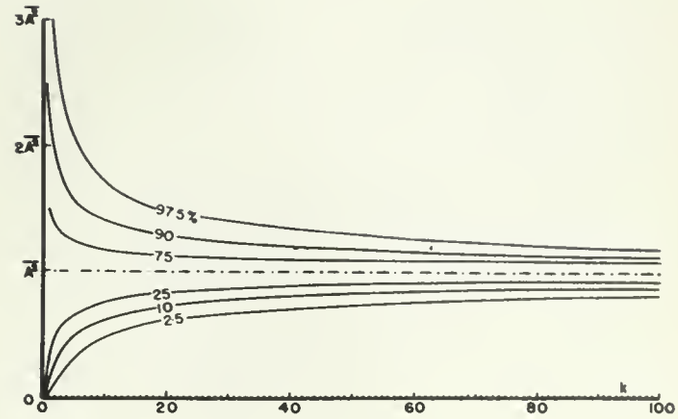

FIG. 110.—LIMITS BELOW WHICH 2.5, 10, 25, 75, 90 AND 97.5% OF THE AVERAGES OF k INDEPENDENT VALUES OF A^2 MAY BE EXPECTED TO LIE.

At k = 40, for instance, 90% of the averages of 40 values of A^2 fall below 1.2 $\overline{A}^2$, 10% below 0.8 $\overline{A}^2$.

($\overline{A^2}/C^2 = 1$), then the long-term average of p/k is about 0.38, and the value of the mean signal intensity calculated will in 68% of the cases lie between 0.87 and 1.13 of the correct value when 100 independent measurements are available.

We have seen how the availability of as many as possible independent signals renders the interpretation of the echo easy and accurate. We shall now be concerned with the question of how these independent signals from the same meteorological condition can be obtained.

The obvious way is to wait. The scatterers reshuffle themselves, if we give them a chance, so that pulses sent out the proper time apart will give rise to practically independent signals. Our friends at M.I.T. are experts in the theory and practical exploitation of this reshuffling process and I will say no more about it, except that at common PRF's (200 to 1000 sec^{-1}) no appreciable redistribution among the scatterers can be expected between successive pulses along the same path. Many of the pulses we are sending out therefore add little to our knowledge. They do, however, effectively improve the signal/noise ratio.

We will now turn to the signal fluctuations resulting from the motion of the pulse out from the radar.

It is convenient in studying the signal trace painted by a pulse going out from the radar to divide the contributing region into a series of n contiguous slices, each contributing an amplitude A_n. The rectangular components of these phasors are Gaussian deviates; their squares are additive, and the sum of the squares is the square of the amplitude of the whole contributing region.

The X and Y components of A_n are similarly additive, and this allows the computation of sample signal sequences. We were able to build up X and Y as the algebraic sums of 100 (for n) deviates which we took from the tables of randomly-sequenced Gaussian deviates of Wold. The addition of one deviate to X and one to Y, and the removal of the 100th-preceding deviates gives new values of X and Y, and so of A, representing the condition existing when the contributing region has moved by 1/100th its own length. After 100 such changes, one has a whole new set of deviates, an independent value of A. In carrying out the computations, we first drew a hodograph, consisting of 100 vectors, each being the vector difference of two terms. Fig. 112 shows the procedure; (a) shows how a signal amplitude (PL 1) builds up from zero (in ten steps corresponding to n = 10 for greater simplicity), (b) how after another similar ten steps the signal amplitude is made over into a new value. At the bottom of the figure are shown the A_n phasors of the twenty slices into which the two consecutive contributing regions were divided. Fig. 113 shows a hodograph of the kind we drew in our work. It corresponds to a divi-

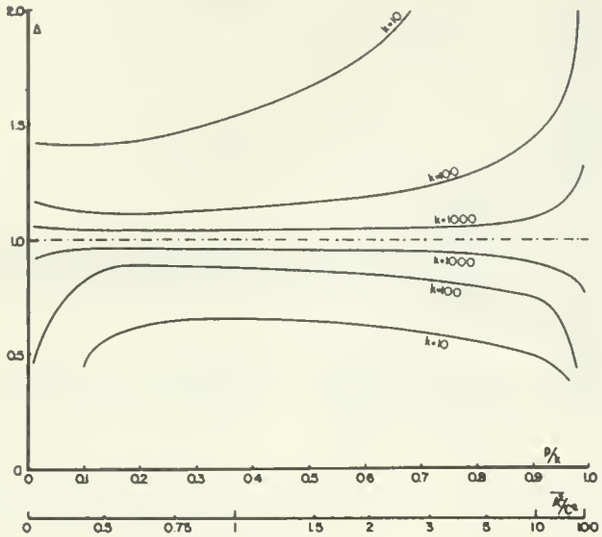

FIG. 111. —FLUCTUATIONS IN A^2/C^2 (AS COMPUTED FROM OBSERVATIONS OF p/k) ABOUT ITS TRUE VALUE, AS A FUNCTION OF k AND THE TRUE VALUE OF p/k.

The pair of lines for each value of k include 68% of the values occurring.

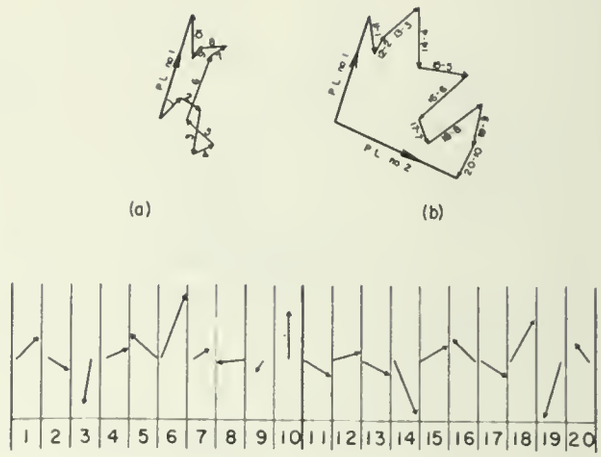

FIG. 112. —SIMPLIFIED HODOGRAPHS.

(a) Half-pulse length PL 1, building up from zero. (b) Building up PL 2 from PL 1, each phasor element being the vector difference between a new and an old a term. Below the hodographs, the twenty slices, each with its a term (slightly reduced), into which the contributing regions for PL 1 and PL 2 were divided.

sion into 100 slices. The only independent points on this picture are the zeroth and the hundredth.

We computed altogether thirty such sequences making up a trace of 3000 signal values. Five of these sequences (of amplitude and phase) are shown at the top and bottom of Fig. 114. It may be noted that this sequence contains only six independent points (lying on the vertical lines); the intermediate ones are dependent on each other in that they are made up of the same components as the two nearest points marked by the vertical lines. The top trace is a theoretical sample of what a radar A-scope would look like if all the circuits were perfectly linear and if the amplifier band width were wide enough to reproduce faithfully to time intervals as short as 1/100 of the pulse duration. Second and third traces represent a suppression of detail. In the latter trace only the independent points, joined by a smooth curve, have been retained.

An inspection of this picture also shows that wherever the A is great, the fluctuations in the phase angle Θ are modest, and vice versa. (This is also brought out in Fig. 117.)

The explanation is simple. Fig. 115 shows the vector increment $\Delta \underrightarrow{A}$, whose addition to one value of the signal amplitude produces the next value. The fluctuations in this increment are small compared to those in $\underrightarrow{A}$. When $\underrightarrow{A}$ is large, therefore, the effect of $\Delta \underrightarrow{A}$ on the phase angle is in general

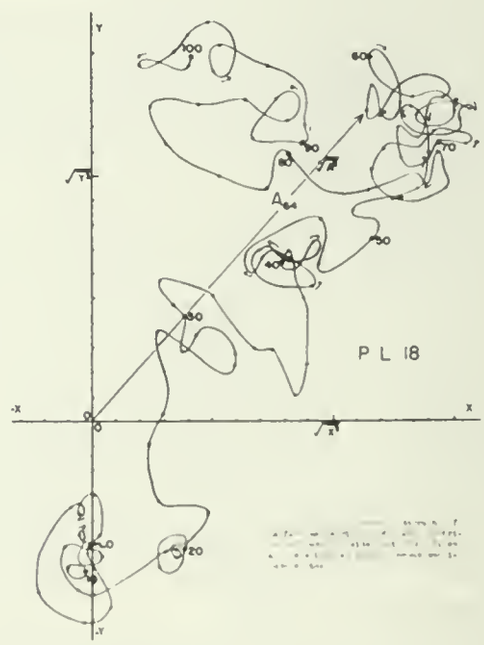

FIG. 113. —TYPICAL HODOGRAPH (OF HALF-PULSE LENGTH PL 18).

Contributing region is divided into 100 slices. A_{64} is a typical instantaneous amplitude vector.

small; when $\underrightarrow{A}$ is small, Θ will change more radically. If the average value of A over a length, say h/2, were corrected according to the extent of the fluctuation in Θ, a much improved value of $\overline{A}$ might be obtained.

We were greatly interested in assessing the value of the information conveyed by the incompletely independent signals—such as make up most of the top trace of Fig. 114. Thus we compared the mean values of A^2 obtained by averaging under the full curves, with that of the means of the instantaneous values separated in range by h/2, i.e., adjacent independent data. The results are shown in Fig. 116, where the thin line is a plot of the averages of from k = 1 to k = 30 independent A^2-values. The thick line shows the running average of all the A^2's computed—i.e., the independent ones plus the 99 semi-dependent ones intervening between successive independent values. On the whole, the curves indicate a slightly more rapid convergence to the true average when the dependent values are included along with the independent ones. No unambiguous conclusions can be drawn from this figure, however. But considerations of the standard deviation point in the same direction and so does a theoretical investigation* on continuous transitions from one independent point to another. The contribution of the nonindependent points in any case seems fairly small, so that with little error we can say that the amount of information which can be extracted from the signal coming from a given range is little better than that contained in the independent signals present in this range interval.

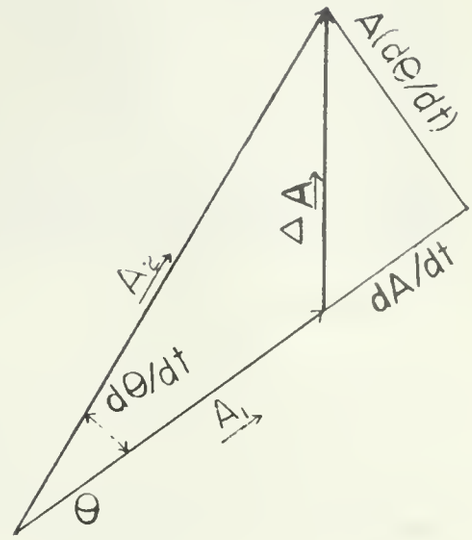

FIG. 115. —VECTOR DIAGRAM.
The diagram shows amplitudes and phases of consecutive (i.e., $\tau/100$ apart in time) amplitude vectors A_1 and A_2.

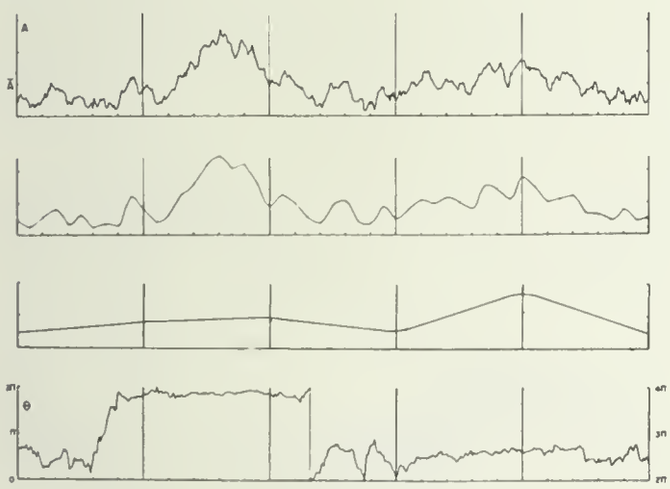

FIG. 114. —TRACES OF COMPUTED SIGNAL AMPLITUDE A FOR FIVE HALF-PULSE LENGTHS.
Top row is hypothetical sample of an ideal A-scope sweep with an amplifier band width $100/\tau$; thus one hundred points are shown per half-pulse length. Second row, traces with 90% of the detail suppressed; these correspond to a band width $10/\tau$. In the third row, only independent points are shown; the necessary band width is $1/\tau$. (τ is the pulse duration.) The bottom trace shows the variation of the phase angle Θ, with detail corresponding to amplitude trace at top.

There is, however, another way of finding independent bits of information. The successive vector variations $\underrightarrow{\Delta A}$ of Fig. 114 are independent of each other. They, rather than the signal amplitudes, should therefore be analyzed. For the case of 99 signals intervening between two independent ones, it would thus appear that we have about 100 times as many independent data. Since the true mean of $(\Delta A)^2$ is known to be a definite fraction of $\overline{A^2}$ (in our case, about 1/200), the latter can be computed with the accuracy otherwise obtainable only from 100 independent measurements of A^2. $\underrightarrow{\Delta A}$ can be resolved into rectangular components, parallel and perpendicular to $\underrightarrow{A}$. Either of these is almost of as much use as $\underrightarrow{\Delta A}$. Sample traces of these quantities are shown in Fig. 117. The absolute value of the component parallel to $\underrightarrow{A}$ could be obtained in practice by differentiation and subsequent full-wave rectification of the video signal. There are, however, two practically undesirable

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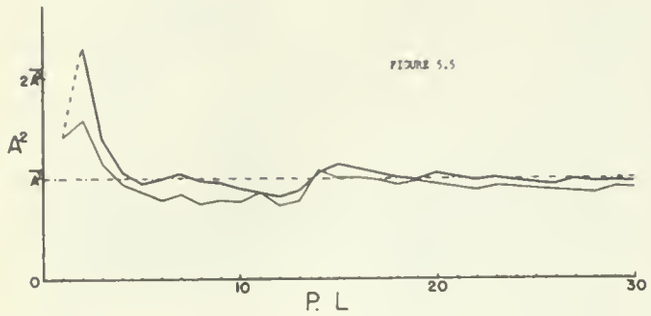

FIG. 116. —COMPARISON OF THE AVERAGES OF (INDEPENDENT) AND (INDEPENDENT + NON-INDEPENDENT) VALUES OF A^2.

The thin line is a plot of the averages of from 1 to 30 independent A^2-values. The thick line shows the running average of all the (A^2)'s computed, i. e., of the independent ones plus the 99 semi-dependent ones intervening between any two successive independent values. The faint background is a copy of Fig. 110 and shows the 5, 20 and 50% confidence limits.

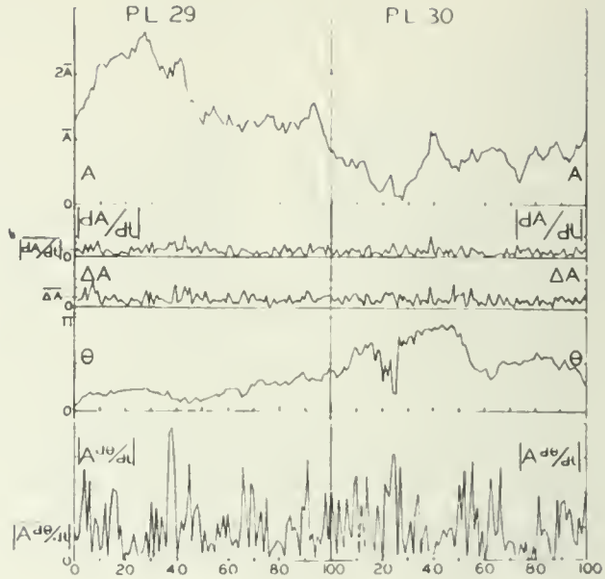

FIG. 117. —TRACES FOR TWO TYPICAL HALF-PULSE LENGTHS OF A, $|dA/dt|$, ΔA, Θ, AND $|A(d\Theta/dt)|$

Each function (with exception of Θ) is plotted in units of its long-term mean. (These quantities are all shown in the vector diagram of Fig. 115.)

characteristics of such a technique. It is necessary to provide for a time constant shorter than the pulse duration which entails an increased band width in the receiver and the consequent increase in background noise level. At the same time, the signal we wish to observe is only a fraction of the full amplitude. For both these reasons we have to countenance a decrease in the signal/noise ratio.

The suggestion of observing $\overrightarrow{\Delta A}$ is therefore exactly equivalent to shortening the pulse duration. This would place the same burden on receiver band width and would similarly weaken the signal intensity; it also would increase the number of independent signals in the same way. One advantage of the short time-constant receiver, which is not shared by the shortened pulse, is that the former could be made variable with range. In this way the excess power at short range could be utilized to provide increased precision.

Let us now look more closely at the appearance of the radar signals on the displays. The one kind of display in common use, which we have already referred to, is the displacement presentation or A-scope. In Fig. 108, on the left, we are showing the probability distributions of the signal for a display of A^2, A, and log A^2. In the central column have been plotted representative A-scope traces of

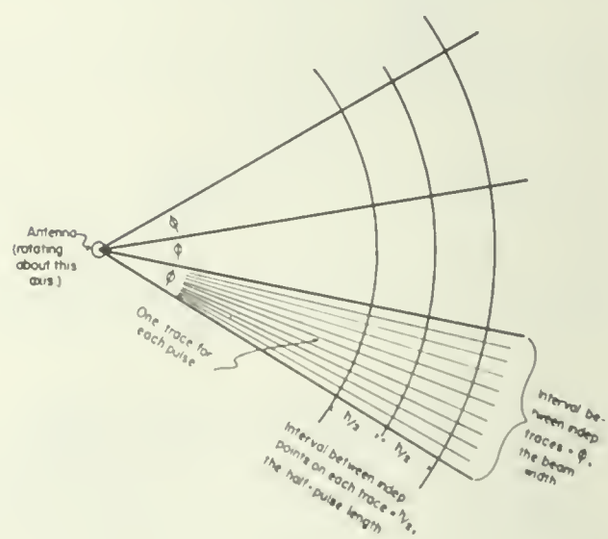

FIG. 118.

Moving along beam through range interval h/2, the half-pulse length, or scanning through ϕ, the beam width, leads to an independent signal value.

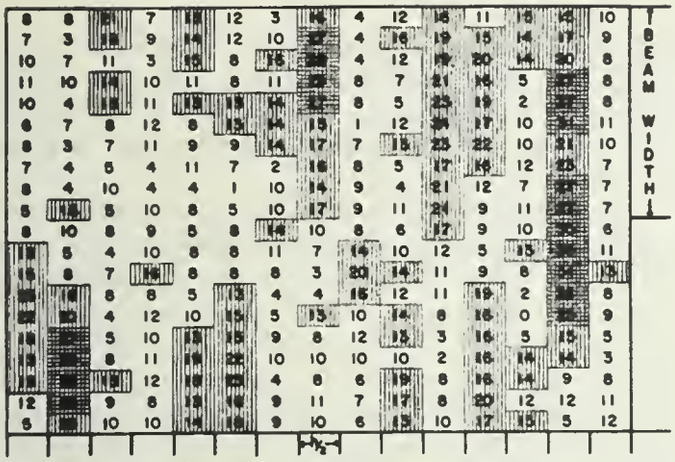

FIG. 119.—TYPICAL APPEARANCE OF A SECTION (15 HALF-PULSE LENGTHS x 2 BEAM WIDTHS) OF A BRIGHTNESS-MODULATED DISPLAY.

The numbers represent amplitude values received from element of area resolved ($h/2 \times \phi$). Vertical shading indicates amplitudes exceeding average ($\overline{A}$ = 12.5); cross shading, amplitudes exceeding 1.5 $\overline{A}$ = 18.75.

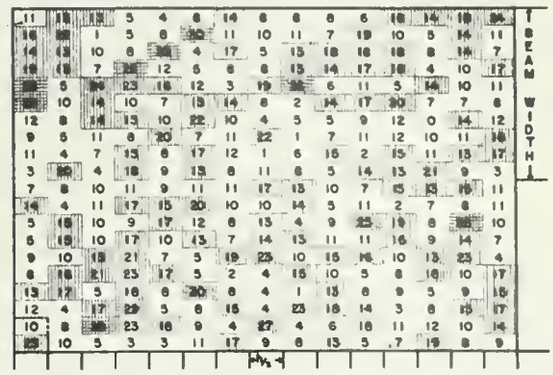

FIG. 120.

Same situation as Fig. 119, except that amplitudes in adjoining rows (as well as in the columns) are independent. Note how the vertical streaking in the background, so predominant in Fig. 119, is largely absent.

independent points. In every case the fluctuations are seen to be very great, indicating once more that a glance at an A-scope is hardly sufficient to estimate a true mean signal value. Single sweeps—particularly their photographs—lend themselves readily to a determination of the fraction of the time spent above a given threshold. The greatest accuracy is obtainable when this fraction is about 20%, and since this fraction varies most gradually on a logarithmic display, the widest range of precipitation echoes can be investigated on a logarithmic scope.

When several sweeps are superimposed, the interpretation is more difficult; when very many sweeps are superimposed, a new approach suggests itself, however. This superposition might be by long-persistence coating or by time-exposure photography. The details of individual lines are lost; instead a continuous gradation of luminance is obtained. Since the response of phosphorescent or photographic material is not usually linear, the probability distributions themselves give no direct clue to the gradation of luminance on the display. If the response is assumed to be logarithmic, however, then it is the logarithm of the probability which is the pertinent function. These functions are plotted in the right-hand column of Fig. 108. It may be seen at once that the A^2-scope is least useful here. The slope of the line could be determined by photometry, of course, and $\overline{A^2}$ could be deduced.

No photometry is required, however, in the case of A and log A^2 scopes. The level of maximum luminance, and so of the most probable value of A or log A^2, can be determined simply by reducing the gain. From the most probable, the mean value can at once be deduced. Among these two displays, the logarithmic one is again to be preferred on account of the somewhat more pronounced maximum in the log probability curve.

The other common kind of display is the brightness-modulated area display. As the antenna moves, usually transmitting many sweeps for every beam width, the signal at a given range will exhibit fluctuations similar to those discussed above for variation in range with a given antenna position. Fig. 118 shows the separation of points both in the direction of the beam and at right angles to it, between independent points. In most existing equipment the limitation of amplifier band width ensures that adjacent signals along the same sweep are practically independent, while signals at the same range, but on successive sweeps rarely are. We can illustrate this by Fig. 119, where we are showing a hypothetical appearance of a section of an area display (PPI or HPI) with the numbers representing amplitudes received from the element of area resolved (which is $h/2 \times \phi$). The vertical and cross shading indicates points where the signal amplitude exceeds its mean, and 1.5 x mean value. Note the vertical streaking in the shading, indicative of lack of independence in each column. The diagram is made up for a radar which transmits 10 pulses while the antenna turns through one beam width. Fig. 120 depicts the same situation with consecutive sweeps completely independent of each other. The streaks in the shading have largely disappeared.

The smallest area of resolution in any case is $h/2 \times \phi$, i.e., half-pulse length x beam width; if the desired area of resolution is greater, then the independent signal values received inside the desired region can usefully be averaged. In many applications this technique itself will not produce

a sufficiently high k-value, i.e., a high enough degree of accuracy. In such cases one can superimpose successive scans. Relative motion of the scatterers is almost certain to provide effective independence for successive scans.

CONCLUSION

Single observations of radar weather echoes are subject to such severe fluctuations as to be of little use for quantitative interpretation. When several independent signals are averaged (according to amplitude or intensity or intensity level), substantial smoothing and corresponding gain in accuracy can be obtained. The same applies when signals are classified according to whether they fall above or below a given threshold.

Averaging the signal traces obtained from a pulse traveling out from the radar is of limited use if it involves averaging of mutually dependent signal values. Shortening the pulse duration (if enough power is available) is a possible way of increasing the number of independent signals from a given interval of range.

When studying the superposition of many traces on an A-type scope, the resulting distribution of brightness on the scope can be interpreted easily to give the mean value of the received intensity, if the presentation is either linear or logarithmic. A square-law presentation is not nearly so suitable.

In brightness-modulated displays the smallest area that can be resolved has the dimensions beam width x half-pulse length. When less precision in defining the location can be tolerated in mapping out the precipitation, this loss of definition can be turned to advantage in reducing the signal fluctuation, since it allows averaging of independent signals from adjoining elemental areas.

DISCUSSION

W. HITSCHFELD. —Mr. Hill opened the discussion by inquiring into the usefulness of the usual techniques, which is the building up of a great many pulses per beam width and the integration of the signals on the scope. It was pointed out in reply that while many pulses were more useful than one (because of the resulting improvement in the signal to noise ratio), substantial advantage could be taken of many signals only if they are all independent. The average value of such signals was significant; the average of a series of mutually dependent ones meant considerably less.

M. H. LIGDA. —I was very interested in this paper. It seems to me that we are going to have to focus more attention on the analysis of the PPI scan to get the areal distribution of rain intensity. We have, with our Pulse Integrator at M.I.T., made an effort to make the signal intensity measurement at a single point. To spread the measurement over an area we are going to have to devise entirely different techniques. I am quite curious to know—have there been any ways suggested to accomplish this actual integration from a PPI scope? The possibilities and problems it presents are very intriguing and quite perplexing. There are one or two things that have troubled me. One is the persistence of the PPI tubes that would spoil the measurements; evidently we have to use a nonpersistent scan. Second, I have the feeling (and here I would really like to get some information) that the present PPI phosphors do not have enough dynamic range to present the variability of the echo which the receivers can accommodate. Does anyone know the dynamic range of a P7 phosphor?

J. S. MARSHALL. —One possible way of treating your problem might be by photography. Of course you photograph only the moving trace, cutting out any effects of the persistence of the phosphorus by a color filter, if necessary.

This photographic integration may involve some high grade photography, but should avoid a lot of expensive circuitry and some of the difficulties connected with unknowns in the tube coating.

THE USE AND LIMITATIONS OF RASAPH

BY AARON FLEISHER*

WITH DISCUSSIONS BY M. L. STONE, A. FLEISHER

The amplitude of radar echoes returned by clouds and precipitation fluctuates irregularly from one pulse to the next. The total return from a group of scatterers contained within half the pulse length of a radar is the vector sum, in phase space, of the individual returns. If the scatterers move along the beam then the phases of their returns will change and, if there is relative motion among them, so will the vector sum. We can expect then, that the rate of fluctuation will provide some information about shear and turbulence in the volume illuminated.

We may regard these fluctuations as a statistical time series and inquire into what information of meteorology interest it can yield. A time series is specified when we know all the probability densities $W_n(x_1t_1, x_2t_2, x_3t_3 \text{ -- } x_nt_n)$, where $W_n\, dx_1\, dx_2 \text{ -- } dx_n$ is the joint probability that the variable x, which in our case will be either the amplitude or power returned from a single pulse, lies in the range $(x_1, x_1 + dx_1)$ at time t_1 and $(x_2, x_2 + dx_2)$ at t_2 --- and $(x_n, x_n + dx_n)$ at time t_n. We shall assume that these probability densities are not functions of the time so that the time origin can be chosen arbitrarily. Therefore, only the differences $t_2 - t_1$, $t_3 - t_1$, etc. are relevant. Such a time series is called stationary.

A probability distribution of order n depends on 2n - 1 parameters and therefore the difficulty of deriving or computing W_n mounts rapidly with n. We shall be concerned only with the second order probability distribution and the moments obtained therefrom. These are the average values of the products $x_1^{\alpha_1}\, x_2^{\alpha_2}$,

$$\overline{x_1^{\alpha_1} x_2^{\alpha_2}} = \int_{-\infty}^{\infty}\int_{-\infty}^{\infty} x_1^{\alpha_1}\; x_2^{\alpha_2}\; W_2(x, x_2t)\, dx_1\, dx_2 \qquad (1)$$

This is an ensemble average which we shall assume to be the same as the time average

$$\langle x_1^{\alpha_1}\; x_2^{\alpha_2} \rangle = \lim_{T\to\infty} \frac{1}{2T} \int_{-T}^{T} x_1^{\alpha_1}\; x_2^{\alpha_2}\, dt \qquad (2)$$

*Research Associate, Weather-Radar Project, Massachusetts Institute of Technology, Cambridge, Massachusetts.

In particular, when $\alpha_1 = \alpha_2 = 1$, the averages yield the autocorrelation function of x(t). The Fourier transform of the autocorrelation function is called the power density spectrum and has actually the dimensions of power per unit of angular frequency when x(t) is a voltage or a current. We shall deal here only with the spectrum; however, an equivalent development can be made in terms of the autocorrelation function.

The second probability distribution of the returned signal was derived by Siegert (1)** with the assumptions that:

- the number of scatterers in the beam is large and that they move with the wind independently of each other;
- the return from one scatterer is independent of the return from the rest and that the amplitude of this return is independent of the time;
- the phases of the individual returns are uniformly distributed.

The expression for the spectrum comes out to be:

$$F(\overline{w}) = \frac{\lambda}{4\pi}\int_{-\infty}^{\infty} S(u)\; S(u + \frac{\lambda\overline{w}}{4\pi})\, du \qquad (3)$$

where

$$S(u) = \frac{1}{\overline{r^2}}\int_{0}^{\infty} r^2\, \overline{\mathcal{W}}(r, u)\, dr \qquad (4)$$

$$\overline{r^2} = \int_{-\infty}^{\infty}\int_{0}^{\infty} r^2\, \overline{\mathcal{W}}(r, u)\, drdu \qquad (5)$$

and $\overline{\mathcal{W}}(r, u)drdu$ is the joint probability that a scatterer moving with a speed in the range (u, u + du) away from the transmitter returns a signal with the amplitude in the range (r, r + dr). S(u)du is then the fraction of the average received power returned by scatterers moving with a speed between u and u + du. The wave length of the radar is λ. The spectrum as thus obtained is normalized, for

**Numerals in parentheses are reference numbers in BIBLIOGRAPHY at end of this paper.

$$F(\overline{w})d\overline{w} = \int_{-\infty}^{\infty}\int_{-\infty}^{\infty} S(u)\, S(u + \frac{\lambda \overline{w}}{4})\, \frac{\lambda}{4\pi}\, du\, d\overline{w} = 1 \qquad (6)$$

We shall make two additional assumptions. First, that the velocity of the scatterer is independent of its scattering cross-section. This is equivalent to the conditions that the scatterers are uniformly distributed in the beam (e.g., there is no bright band); that they do not tumble in their motion; and that they move directly with the wind. This assumption will not apply when the beam is elevated to large angles, for then an appreciable component of the motion along the beam will be due to fall velocities which are a function of the radius and therefore of the scattering cross-section. Then

$$W(r, u) = W_r(r) \cdot W(u) \qquad (7)$$

With this assumption equation (3) becomes

$$F(\overline{w}) = \frac{\lambda}{4\pi}\int_{-\infty}^{\infty} W(u)\, W(u + \frac{\lambda \overline{w}}{4\pi})\, du \qquad (8)$$

An immediate consequence is that the spectrum is independent of the drop size distribution and therefore, since the spectra are normalized, they are all directly comparable.

The wind that enters into equation (8) is the total wind away from the transmitter, which shall be written as the sum of the mean wind, as measured by a balloon ascent, and a residue, the turbulent wind.

$$u = \overline{u} + u' \qquad (9)$$

In terms of the distributions of $\overline{u}$ and u', $W^{I}(\overline{u})$ and $W^{II}(u')$, the distribution of u is

$$W(u) = \int_{-\infty}^{\infty} W^{I}(\overline{u})\, W^{II}(u - \overline{u})\, d\overline{u} =$$

$$\int_{-\infty}^{\infty} W^{II}(u')\, W^{I}(u - u')\, du' \qquad (10)$$

When equation (10) is inserted into (8), then

$$F(\overline{w}) = \frac{\lambda}{4\pi}\int_{-\infty}^{\infty} W^{I}(\overline{u})\, d\overline{u} \int_{-\infty}^{\infty} W^{I}(v)\, dv$$

$$\int_{-\infty}^{\infty} W^{II}(u - \overline{u})\, W^{II}(u + \frac{\lambda w}{4\pi} - v)\, du \qquad (11)$$

or

$$F(w) = \frac{\lambda}{4\pi}\int_{-\infty}^{\infty} W^{II}(u')\, du' \int_{-\infty}^{\infty} W^{II}(v)\, dv$$

$$\int_{-\infty}^{\infty} W^{I}(u - u')\, W^{I}(u + \frac{\lambda w}{4\pi} - v)\, du \qquad (12)$$

These equations contain functions of three quantities: the spectrum, the mean wind, and the turbulent wind. The spectrum can be measured; this problem will be taken up in a later section. There are various devices for obtaining the mean wind. The additional information, therefore, that these fluctuations yield is the distribution of the turbulent wind in the free atmosphere. The details of the derivations and the conditions under which a unique solution to equations (11) and (12) exists will be discussed in a forthcoming Technical Report of the Massachusetts Institute of Technology Weather Radar Research Project.

For the purpose of illustration, assume that the turbulence is Gaussian and that the mean wind across the beam is constant. Then

$$W(u') = \frac{1}{\sigma\sqrt{2\pi}}\, e^{-\frac{u'^2}{2\sigma^2}} \qquad (13)$$

$$W(\overline{u}) = \delta(u_o - \overline{u}) \qquad (14)$$

$\delta(x)$ is the delta operator of x.

$$\delta(x) = \begin{cases} \infty & \text{if } x = o \\ o & \text{if } x \neq o \end{cases} \quad \text{and} \quad \int_{-\infty}^{\infty} \delta(x)dx = 1$$

When these are substituted into (12), the spectrum is

$$F(\overline{w}) = \frac{\lambda}{2\sigma\pi^{3/2}}\, e^{-\frac{1}{4\sigma^2}\frac{(\lambda\overline{w})^2}{(4\pi)^2}} \qquad (15)$$

which is a normalized Gaussian curve, with a variance of $\frac{4\pi\sigma\sqrt{2}}{\lambda}$. If the mean wind were constant and there were no turbulence, the spectrum would have the form $F(\overline{w}) = \delta(\overline{w})$. The presence of turbulence broadens the spectrum to an extent that is a function of the width of the distribution of the turbulent wind.

The spectrum corresponding to various non-turbulent wind regimes have been computed by Hilst (2), and Chatfield (3) has attempted to relate the measured spectrum to the observed wind shear under the assumption that the turbulent component is zero.

MEASURING THE SPECTRUM

A radar signal spectrograph dubbed "Rasaph" was built by the Weather Radar Research Project.

The output of Rasaph depends on its electronic characteristics and the length of the observation interval. It will depend also on the statistical properties of the input, specifically on the nature of the probability distributions and the fact that a random function extends indefinitely in time in both directions. The output will be composed of two terms, the transient and the steady state responses. An expression for the mean square fluctuations of the output has been obtained by Johnson and Middleton (4).

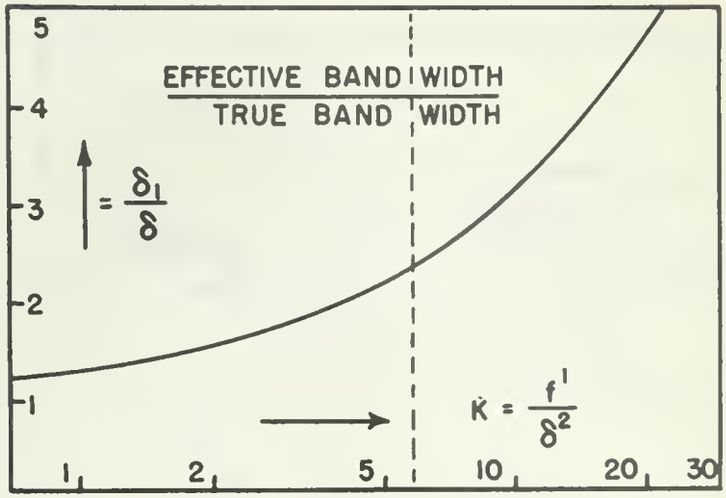

FIG. 122.—INCREASE IN EFFECTIVE BAND WIDTH WITH INCREASE IN SPEED IN CONTINUOUS ANALYSIS, THE NOMINAL BAND WIDTH BEING PRESCRIBED.

$$\sigma^2(T) = \int_0^\infty\int_0^\infty \left[\psi_z(u-v) - \overline{Z}^2\right] h(u)\,h(v)\,du\,dv - 2\int_T^\infty\int_0^\infty \left[\psi_z(u-v) - \overline{Z}^2\right] h(u)\,h(r)\,du\,dv$$

$$+ \int_0^\infty\int_0^\infty \left[\psi_z(u-v) - \overline{Z}^2\right] du\,dv\;\; h(u)\,h(v) \qquad (16)$$

Z(t) is the input.
ψ_z is the autocorrelation of the input.
h(u) is the weighting function of Rasaph's filter.
T is the observation interval.

The first term derives from the steady state response to the fluctuating signal. The remaining two derive from the transient response. If the observation interval were infinitely long, then these last two terms would vanish. The first term, however, remains and, therefore, oscillations in the output will always be with us.

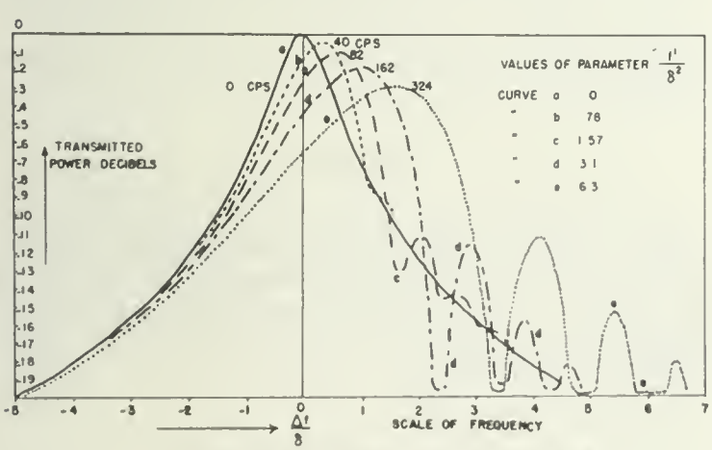

FIG. 121.

Let us consider first the transient error. Rasaph has been described in detail by Williams (5). For the present purposes it may be regarded as a two-cycle bandwidth variable filter scanning the frequency interval from 3 to 300 cps in one minute at a rate that is proportional to the frequency; whence the rate of scan is 0.077f (f being the frequency). The time constant of a filter is of the order of magnitude of 1/bandwidth. The time that Rasaph spends in the vicinity of any particular frequency is 2/0.077f. Equating this to 1/bandwidth, we see that the response at all frequencies greater than 50 cps is infected with transients. The scanning rate is too fast and, therefore, the sampling time too short.

To estimate these errors, we shall borrow from Barber and Ursell's (6)(7) analysis of the transient response of a varying filter to a fixed signal; or equivalently, the response of a fixed filter to a frequency modulated signal. Fig. 121 is a family of universal resonance curves showing the change in transmission of a simple resonant circuit during a continuous frequency analysis. The ordinate is the transmitted power in decibels with reference to the peak transmission when the system is stationary. The abscissa is scaled in the difference between a fixed and variable frequency in multiples of the

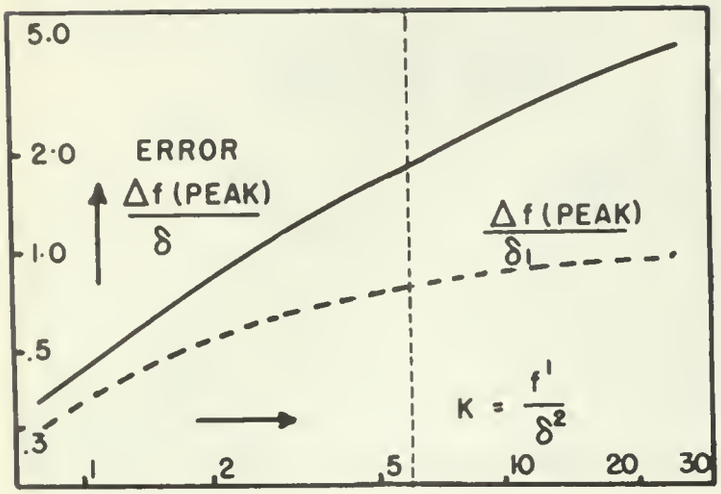

FIG. 123. — FREQUENCY LAG IN PEAK TRANSMISSION IN CONTINUOUS ANALYSIS AS A FRACTION OF THE NOMINAL BAND WIDTH (FULL LINE) AND AS A FRACTION OF THE EFFECTIVE BAND WIDTH (BROKEN LINE).

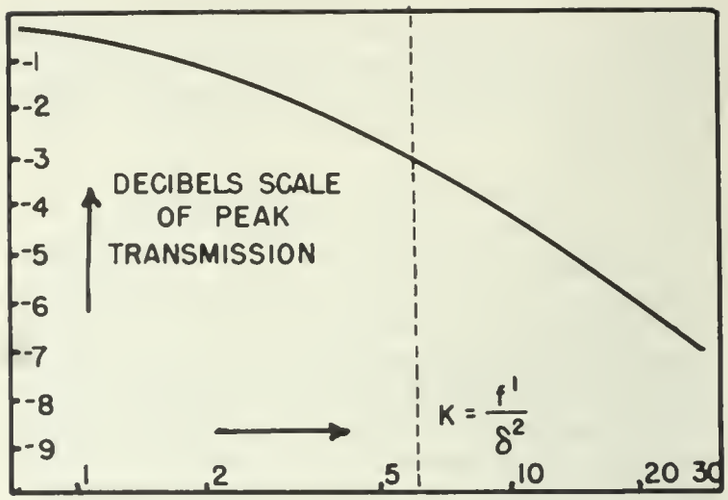

FIG. 124. —POWER TRANSMITTED BY A FILTER IN CONTINUOUS ANALYSIS.

bandwidth δ. The curves are labeled in values of the parameter f'/δ^2, where f' is the scanning rate. Curve (a) represents the response when the frequency is stationary and curves (b) to (e) the response for increasing rates of scanning. The transmission increases smoothly but decays in a series of beats which grow more pronounced as the rate of scan increases. This beating apparently results from the interference between the output when the signal has passed the resonant frequency and the free oscillation of the filter deriving from the energy stored at resonance. Since Rasaph's rate of scan is a function of frequency, its transmission characteristics cannot be summarized by any one curve. The curves are labeled with the frequency at which they apply to Rasaph. The transient errors are evident:

- the peak transmission is too small;
- the resonant frequency has shifted, the sign of the error depending on the direction of scanning;
- the true band width of the filter is increased because spurious overtones are present.

These errors are individually plotted against the parameter f'/δ^2 in Fig. 122, 123 and 124; the vertical line passes through that value of f'/δ^2 which corresponds to the maximum scanning rate. The estimates of these errors are not to be taken quantitatively, because they were derived for a simple resonant circuit. Rasaph's filter is more complicated.

The transmission errors are of no immediate concern, for they are no worse than the best calibration possible at present. A correction could be applied for the frequency shift. The most serious errors are those that result from the beating and loss of resolution. These errors can be eliminated if the sampling is long enough to establish the steady state response. To avoid the possibility that meteorological conditions may change during the sampling interval, then measurements will be made in one of two ways. The signal is fed into a bank of fixed filters simultaneously so that each frequency has the benefit of the entire sampling interval; or it is recorded on a tape and then run through the scanning filter in a continuous loop at a much reduced rate of scan.

We must still account for the steady state oscillations. Referring again to equation (16), we see that the variance of these oscillations depends on the statistical properties of $\Psi_z(u - v) - \overline{Z}^2$. When these are known, it is theoretically possible to adjust the weighting function $h(u)$ so that the variance is a minimum. At present we have some of these autocorrelation curves but not enough to establish any pervasive pattern.

SUMMARY

The spectrum of the returned signal fluctuations is a datum from which can be obtained a measure of gustiness in the free atmosphere. If it is to be measured with a filter, the sampling interval should be long enough to establish the steady state response of the filter.

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DISCUSSION

M. L. STONE. —I think we might add a little note in passing here that one of the things that affect the variations is on the fluctuation of the frequency during the pulse. Another source of the variations may be due to the use of a noncoherent receiver. We are planning to investigate this.

A. FLEISHER. —In all probability what we want to believe is the correlation. We think that the most information that can be obtained from this study is to give the distribution of the turbulent velocity and the turbulent velocity that can give you the first part distribution. The whole autocorrelation curve is evolved in that work, and it is true it is approximated by that straight line up to where it falls off to zero. The ultimate purpose is not to obtain a point of independence; the point of independence is interesting but that is not the whole story.

UNIDENTIFIED. —Would there be an unknown amount of turbulence?

A. FLEISHER. —Yes, that is right. You cannot get the spectrum of turbulence by studying the autocorrelation. Actually, you are going to need a third or fourth distribution of correlation and information, meteorologically speaking, but with a second order probability distribution and the fluctuation, all you get is the distribution of the turbulent velocity. If you want a turbulent spectrum which is the second probability distribution, you will need a higher complement distribution of turbulent velocity.

MICROWAVE SCATTERING FROM NONSPHERICAL HYDROMETEORS

BY DAVID ATLAS*

INTRODUCTION

In almost none of the work on the reflection (or scattering) of electromagnetic waves from atmospheric hydrometeors has there been a serious attempt to consider quantitatively the scattering from nonspherical particles. Ryde (1)** just mentions it in passing. Of the early workers, I believe only our colleagues Austin and Bemis (2) gave this problem any thought. In their well-known "bright band" paper they noted that flattened particles could return a rather strong signal under favorable orientation. Unfortunately, they left it at that. Last year, Kerker and Hitschfeld at McGill University really got to work on the problem and came up with some very interesting results; but they too stopped prematurely without fully exploring all the scattering characteristics of interest to the microwave meteorologist. It was their paper, however, which gave me stimulus to work on this problem.

The failure to consider this problem earlier may be attributed perhaps to a lack of a rigorous general theory for scattering from nonspherical particles. Although the mathematical formulation of the problem is in essence straightforward, the solution is generally so complex that as far as is known, only very special cases have been solved rigorously. One such solution, for the case of a prolate spheroid being struck head-on, was solved by Schultz (3) at the University of Michigan. Fortunately, however, as in the case of Rayleigh scattering from spherical particles, certain approximations appear to be reasonable when the particles are small in comparison to the wave length. It is for this case that Gans (4) developed his theory of scattering from ellipsoids in 1912.

The work is based on Gans' theory. So was that of Kerker and Hitschfeld (5). Although it has never been experimentally verified, it gives results which are physically reasonable. Gans starts right off by assuming that the scattering from an ellipsoid can be synthesized from three orthogonal dipole moments, each of which is parallel to one of the axes of the ellipsoid. By analogy with spherical particles, it seems reasonable that the effects of the higher order multipoles may be neglected as long as the dimensions of the particles are small in comparison to the wave length. This would appear to be the fundamental limitation of his work. It no longer applies when the particles become electrically large. Just where this is, I do not know. Indeed, this is one of the important problems which need investigation. How accurate is Gans' theory and over what range does it apply? Perhaps Schultz' special case can be used to answer these questions.

QUALITATIVE DESCRIPTION OF THE THEORY

I shall not bore you with a detailed description of Gans' theory, but a qualitative description of its important features is necessary for an understanding of the results. I hope that this first slide (Fig. 125) will give you a mental picture of what happens when a plane polarized wave strikes an ellipsoidal particle. In order to make the picture reasonably clear, we have used a horizontally oriented prolate spheroid or needle whose direction is arbitrary in the horizontal plane. For complete generality, its orientation should have been oblique to the horizontal plane as well. The plane in which the particle is oriented is clearly marked as such. The plane of polarization designates the plane formed by the direction of polarization and the direction of propagation. Now, the heavy solid vector marked E ly-

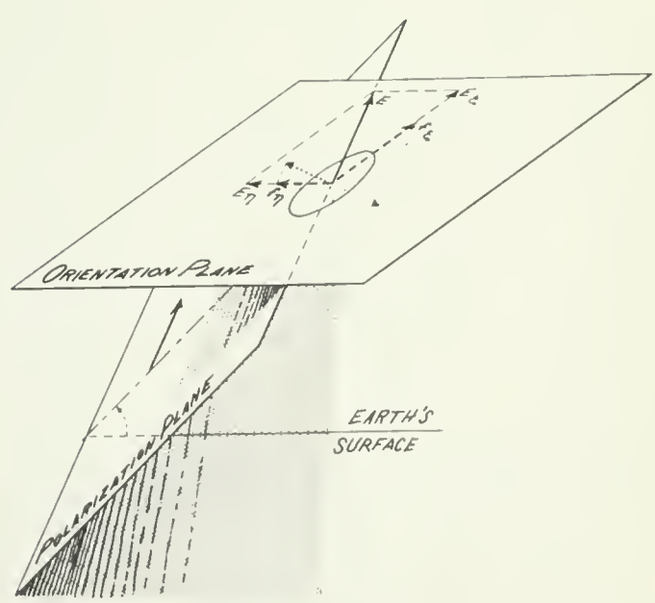

FIG. 125.—VECTOR DIAGRAM ILLUSTRATING THE SCATTERING FROM A HORIZONTALLY ORIENTED PROLATE ELLIPSOID.

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**Numerals in parentheses are reference numbers in BIBLIOGRAPHY at end of this paper.

ing along the intersection of the two planes represents the incident electric field. Following Gans, we first resolve this vector along the directions of the axes of the ellipsoid, giving E_ξ and E_η. There is, of course, no component along that axis which is perpendicular to the plane of orientation. We may now consider each one of these components to act, separately; each one excites a dipole moment in the same direction as shown by the heavier dashed vectors, marked f_ξ and f_η. Now the important feature of Gans' theory is that these dipole moments, which are responsible for the scattered radiation, are proportional to their respective components of the incident field through a factor determined only by the shape and dielectric constant of the material. Both components are also directly proportional to the volume of the particle. We shall discuss these proportionality factors in greater detail. At the moment, suffice it to say that the proportionality factors are larger, the larger the dimension of the axis, and the differences between the factors for the two axes will increase with index of refraction. This is to say that given the same incident field, the dipole moment along the figure axis of the needle will be greater than that along the other two symmetrical axes, although not in direct proportion to their dimensions. The dipole moments will be more sensitive functions of the particle dimensions, the larger the index of refraction.

We are not yet finished. The components of the scattered field are directly proportional to the dipole moments so we may consider these moments to represent the scattered field. However, since a plane polarized antenna can accept only radiation having the same polarization, we must resolve the components of the scattered field along the direction of the incident electric field. This is easily done although we have not shown it on the diagram lest we confuse the picture. Finally, in order to get all the information out of the scattered signal, we should like to know how much radiation is scattered perpendicularly to the plane of polarization. Thus, we have to resolve the components of the scattered field also along the line perpendicular to the polarization plane, giving us the two dotted vectors. Their vector sum represents the cross-polarized component of the scattered radiation. The total intensity scattered by such a particle is proportional to the sum of the squares of the components of the scattered field in the directions parallel to and perpendicular to the direction of polarization. Now this is the intensity scattered by a single particle. If there are a large number N of particles distributed randomly, the total intensity is N times the average intensity from a single particle, where the average is taken over all possible orientations of the particles. Of course, if the particles are not uniform in size or in degree of ellipticity, the averaging must be performed over the distributions of both these variables as well and the problem becomes rather complex.

Before we leave this important slide, let me point out a few of its other features. Suppose we were to rotate the particle around in the plane of orientation so that its figure axis becomes parallel to the incident field. We see that, in this event, only a single dipole moment is excited and that the scattered field is entirely parallel to the direction of polarization. Thus, there is no cross-polarized component. Indeed, whenever the plane of polarization is transverse to one axis of the particle, there is no cross-polarization in the scattered field. Such is the case with horizontally oriented oblate spheroids or plates; they never return any cross-polarized signal. Note also that, in this case, when the radar beam is horizontally polarized, the induced moment and the scattered field are independent of the antenna elevation angle. That is, the disposition of the incident electric field on the horizontal plane is the same regardless of elevation angle. Of course, this is not so when the beam is vertically polarized. Thus, when particles are horizontally oriented, the scattered signal is independent of elevation angle with horizontal polarization and a function of this angle with vertical polarization. This immediately gives us a method for checking the state of orientation of nonspherical particles.

Before going any further, I think a few words about the proportionality factors for our dipole moments would be in order. As I have already noted, the dipole moments induced are proportional to the components of the incident field through a shape factor, as follows:

$$f_\xi = gE_\xi \; ; \quad f_\eta = g'E_\eta \; ; \quad f_\zeta = g'E_\zeta \; , \tag{1}$$

where ξ represents the direction of the figure axis, and η and ζ represent the other two symmetrical axes. These g factors are given by the equation,

$$g = m_o^2 \frac{3V}{4\pi} Le^{-i\psi} = m_o^2 \frac{V(m'^2 - 1)}{4\pi + (m'^2 - 1)P} \tag{2}$$

The value of g' is gotten by replacing P by P', where

$$P' = \frac{4\pi - P}{2} \tag{3}$$

Here m_o is the refractive index of air, m' is that of the particle relative to air, V is the volume of the particle, and P is a factor depending only on the eccentricity of the particle. P is the only shape factor. Over any reasonable range of eccentricity, both P and P' are fractions of 4

The important things to notice in this equation are:

(1) The dipole moments are directly pro-

portional to the volume of the particle. For spheres, therefore, the scattered intensity is proportional to the square of the volume or the sixth power of the radius, which is what we get with Rayleigh scattering.

(2) Notice also that the factor P comes into consideration through the proportionality factor $(m^2 - 1)$. Thus the importance of shape is in direct proportion to the difference of the dielectric constant from 1. This tells us immediately that water with a dielectric constant of about 60 should show up the shape effects much more strikingly than ice whose dielectric constant is about 3. By the same token the effect of wave length in changing the shape of the g curves (not their magnitude) increases with increasing values of P. This effect applies to water only since the index of refraction of ice is very nearly independent of wave length.

(3) Another point which is worthy of note is the fact that g and g' are generally complex when the index of refraction is complex. All this means physically is that the oscillating dipole moments are made to lag the incident field in time. As far as the value of the scattered field is concerned, however, the effect is hardly noticeable, except at 1 cm., where the index of refraction for water has a large imaginary component.

(4) Finally, it is evident that the magnitude of g is larger for water than for ice. For a sphere with $P = 1/3$, the ratio of g for water to that for ice is about 2.4, resulting in a ratio of approximately 5.6 to 1 for the scattered intensity from water over that from an equal ice sphere (at 10 cm.). This is slightly different from the ratio of 5 given by the Rayleigh scattering theory.

Now I mentioned that the effect of shape increases in importance with index of refraction. This is of great consequence when we consider snowflakes of low apparent density, so it deserves further attention.

A rather well-known law in electromagnetic theory is the Lorenz-Lorentz law (6),

$$\left[\frac{m^2 - 1}{m^2 + 2}\right] \frac{1}{\rho_0} = \text{constant},$$

where ρ_0 is the density of the material.

Using this law, we find that if we consider a snowflake to be a low density ice particle, say as low as .05 gm./cm.^{3}, then its dielectric constant (m^2) is only 1.07 as opposed to 3.06 for ice of density 0.9. Since the effect of shape comes into play only in so far as the dielectric constant differs from 1, such low density particles act very much like spheres of the same volume regardless of their shape. The only question in my mind is whether or not a snowflake can be replaced by a low density ice particle. If it is composed of a large number of individual crystals, as many of them are, so that it is more or less a homogeneous mixture of air and ice, it would appear that this effect is bound to occur. Where the flake is composed of only a few crystals, I believe that its scattering properties would be those of an ice particle of complex shape.

REVIEW OF RESULTS

Well, we seem to have gone into the theory in a little greater detail than I had anticipated, so let us get on to a quick review of the results.

Fig. 126 shows us what to expect in the way of scattering from randomly oriented water spheroids. Whether or not nonspherical water particles ever occur on a large scale is another question. However, very much the same results should be observed for melting ice and snow particles. On this graph, the ordinate shows the scattered intensity relative to a sphere of the same volume. The abscissa is axial ratio. On the left we have oblate particles; on the right, prolates. In this case, the particles are randomly oriented. We see that randomly oriented oblate water particles of axial ratio 0.1 can return as much as ten (10) times the signal to be received from equal spheres at a wave length of 10 cm. and about 8 times as much at 1 cm. Prolate water particles of the same axial ratio return as much as 25 times the signal from equal spheres at a wave length of 10 cm. Of course, such distorted water drops can never really be expected except perhaps, as I have indicated, as melting ice

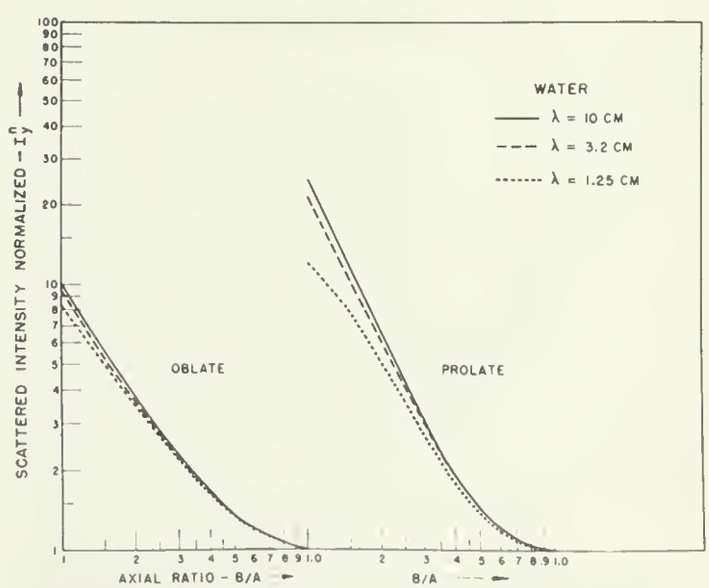

FIG. 126. —VARIATION OF SCATTERED INTENSITY.

Illustrated is the variation of scattered intensity with axial ratio for oblate and prolate spheroids of water at wave lengths of 1.25, 3.2, and 10 cm. (normalized with respect to the radiation scattered from a sphere of the same volume).

particles.

Fig. 127 shows the analogous results for ice particles (not snowflakes). Notice the difference in scale. Even in the limiting case of zero axial ratio, randomly oriented prolate ice particles return only 1.3 times the signal from equal spheres, while corresponding oblate particles scatter only 1.8 times as intensely. Thus, when ice particles are randomly oriented, we can expect but a small increase in signal intensity over equal spheres. For snowflakes of low apparent density, there would be practically no increase. By the way, notice the lack of any sensitivity to wave length.

We come now to depolarization. Fig. 128 shows the ratio of signal intensity in the cross-polarized component to that in the parallel component. On the left, we have water; on the right, ice. Notice the difference in scale; on the left, the ratios go up to 50%; on the right, only up to 5%. Randomly oriented water oblates of axial ratio 0.1 return about 10% depolarization at all wave lengths, while corresponding ice particles return only 3%. Prolate water particles of the same axial ratio return as

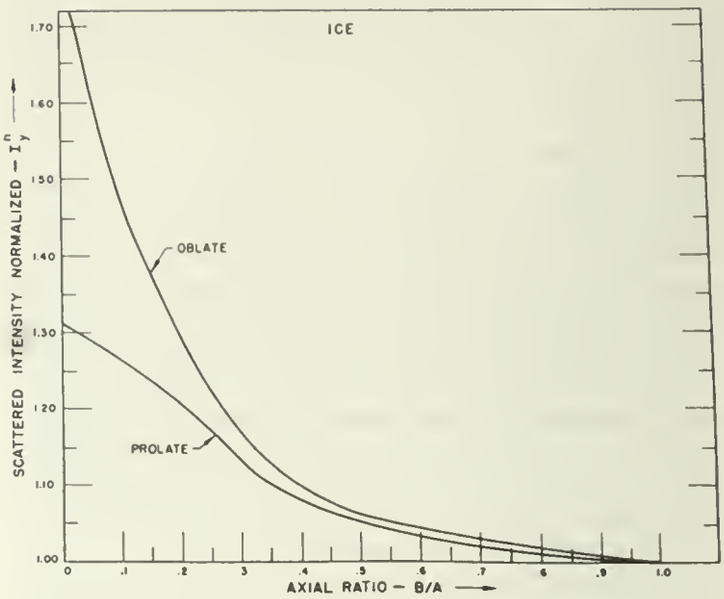

FIG. 127.
Same as Fig. 126 except for ice at all wave lengths from 1 - 10 cm.

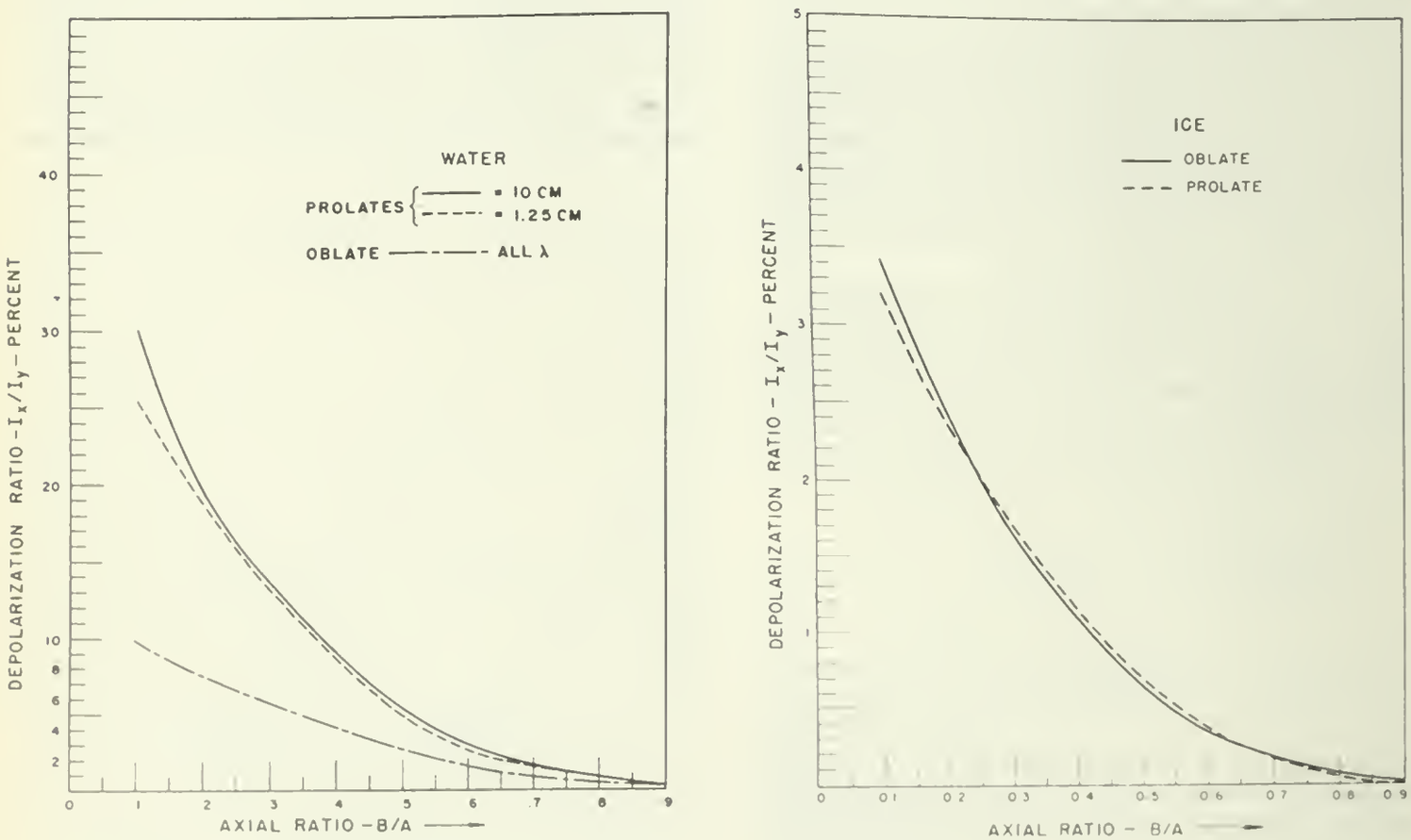

FIG. 128.—THE VARIATION OF DEPOLARIZATION RATIO WITH AXIAL RATIO FOR OBLATE AND PROLATE SPHEROIDS OF WATER (ON THE LEFT), AT WAVE LENGTHS OF 1.24, 3.2, AND 10 CM., AND FOR ICE FROM 1 - 10 CM. (ON THE RIGHT).

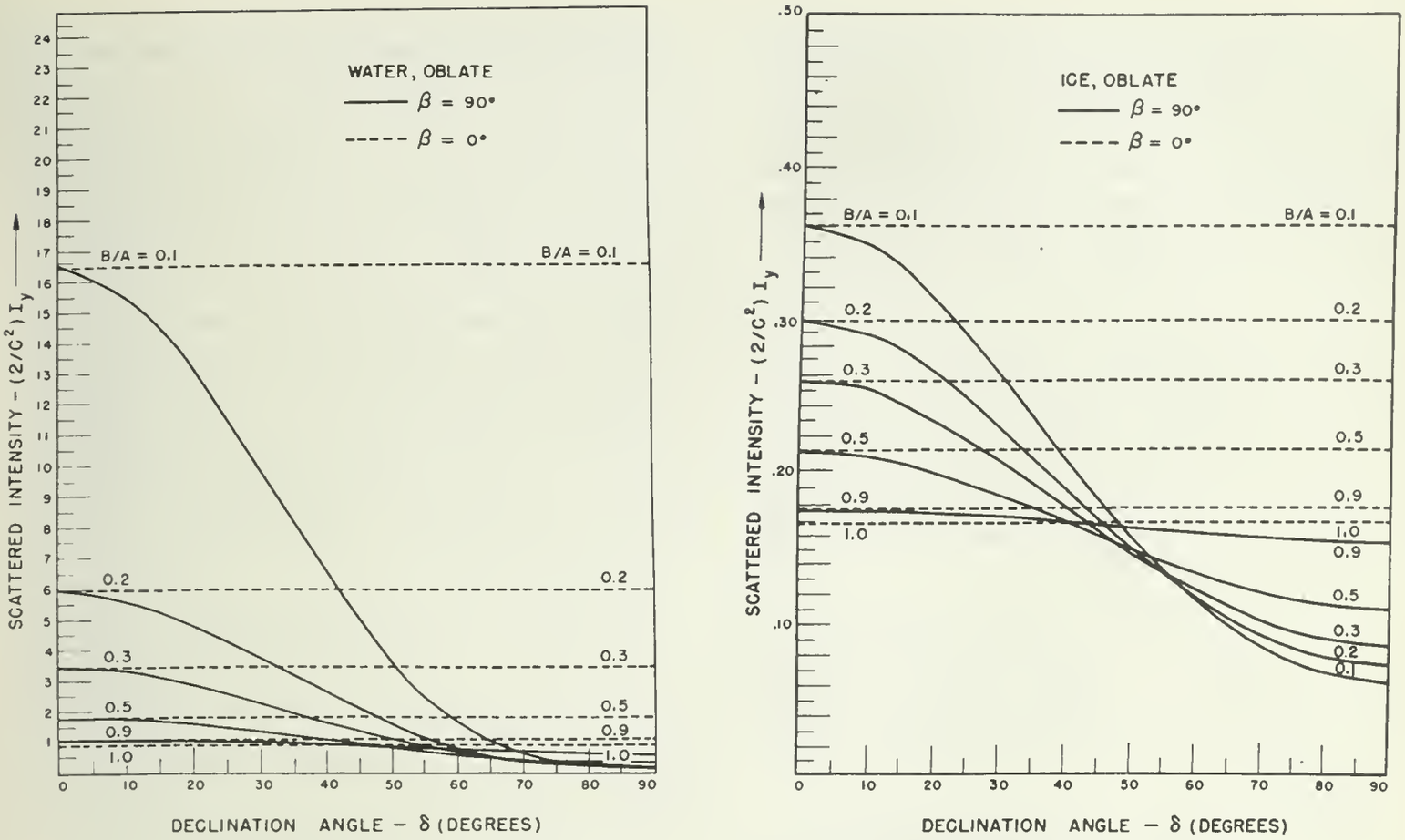

FIG. 129.—THE VARIATION OF THE PRIMARY COMPONENT OF THE SCATTERED RADIATION WITH DECLINATION ANGLE FROM THE ZENITH FOR HORIZONTALLY ORIENTED OBLATE PARTICLES AT VARIOUS AXIAL RATIOS. LEFT, WATER AT 10 CM.; RIGHT, ICE 1-10 CM.

much as 30% depolarized, while similar ice particles only return about 3%. Low density snowflakes return a negligible depolarized component, in keeping with our prediction that they scatter like equal spheres in almost all respects. Again, note that there is a very small difference with wave length, practically none for the oblates, and little for prolates even at the smaller axial ratios. By the way, these results for randomly oriented particles were found earlier by Kerker and Hitschfeld, except for the effects of varying wave length.

Now, let's see what happens when we give the particles some orientation. Fig. 129 shows us oblate particles perfectly oriented in a horizontal plane; that is, the plane of the plates is horizontal. Again, on the left, we have water particles as seen at a wave length of 10 cm., on the right, ice particles. However, the abscissa now represents the angle of declination from the zenith, or the complement of the elevation angle. On the left, the beam is pointing straight up; on the right it is directed horizontally. I shall speak of horizontal and vertical polarization, which describes the state of polarization of the electric vector when the beam is directed horizontally. The various curves are for different axial ratios.

When vertical polarization is used (the solid curves), the signal intensity is largest when the particles are viewed directly from below; the largest dipole moments are excited, and all the scattered radiation is resolved in the proper direction. As the beam is tilted toward the horizon, the signal drops until finally, at the horizon, the intensity is less than that to be received from an equal sphere, which, by the way, is represented by the horizontal line at the bottom having a value of approximately unity for water and about 0.17 for ice. With horizontal polarization, however, the signal remains constant regardless of elevation angle, as shown by the dashed horizontal lines.

As for the magnitudes, the effects are much more pronounced with water than they are for ice. Water oblates of axial ratio 0.1 scatter about 18 times as intensely as equal spheres when viewed from below and only 1/6th as intensely when viewed from the side, with vertical polarization. For ice, the corresponding figures are about 2.2 and .37. This gives us a range of signal intensity of about

6 to 1 with elevation angle in the case of ice. With horizontal polarization, of course, the signal intensity is independent of elevation angle. It is quite possible that this phenomenon is at least partly responsible for the relatively rapid disappearance of echoes from above the freezing level as the antenna is tilted toward the horizon. I should like to point out once more, however, that these effects are small, if not negligible, with snowflakes of low density.

As I mentioned earlier, horizontally oriented oblate particles display no depolarization. In other words, their symmetry and orientation are such that the plane of polarization always cuts transversely to one axis of the particles, thus failing to excite the perpendicular dipole moment. This characteristic might be helpful in identifying horizontally oriented disks, although all spheres and low density snow particles also fail to return a cross-polarized signal.

We come now to consideration of prolate ellipsoids, which we use to represent needles. In a calm atmosphere, it is believed that such particles should fall with their long axis horizontal, but with random direction in the horizontal plane. At least, this is the case for which we have computed the scattering characteristics shown in Fig. 130.

The left-hand graph is for water; the one on the right, for ice. The similarity to the previous case is striking. Here, however, the spread in the curves for water is greater than in the oblate case, while the spread for ice is less. This is due primarily to the greater variation of the dipole moments with axial ratio over the range from 0.1 to 1 for water prolates. For water, as a vertically polarized beam is tilted from the zenith to the horizon, the signal from needles of axial ratio 0.1 varies from 45 times to only 0.47 times as intense as that from spheres of equal volume. For ice, the corresponding variation is from 1.6 to 0.7. For low density snow, of course, there is practically no variation. From the shape of the curves alone, however, we should expect very much the same behavior with horizontally oriented needles as we do with plates.

The major difference between needles and plates comes when we consider depolarization, which is shown in Fig. 131. On the left is the case for water; on the right, ice. Here, we plot the ratio of the cross-polarized component to the parallel component of the scattered radiation. By mistake, we have plotted half the actual values so that the scales

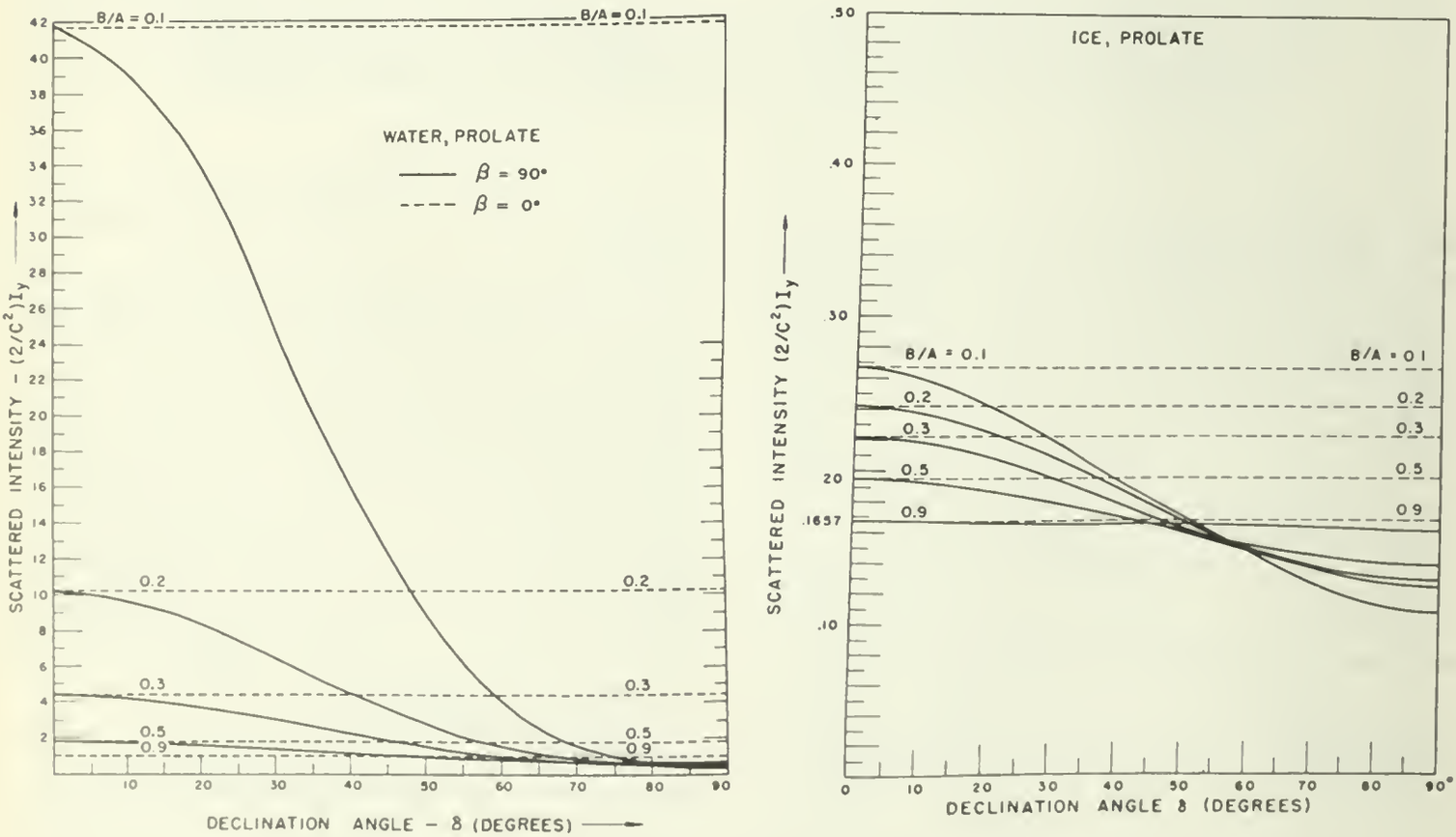

FIG. 130. —THE VARIATION OF THE PRIMARY COMPONENT OF THE SCATTERED RADIATION WITH DECLINATION ANGLE FROM THE ZENITH FOR HORIZONTALLY ORIENTED PROLATE SPHEROIDS. LEFT, WATER AT 10 CM.; RIGHT, ICE 1 - 10 CM.

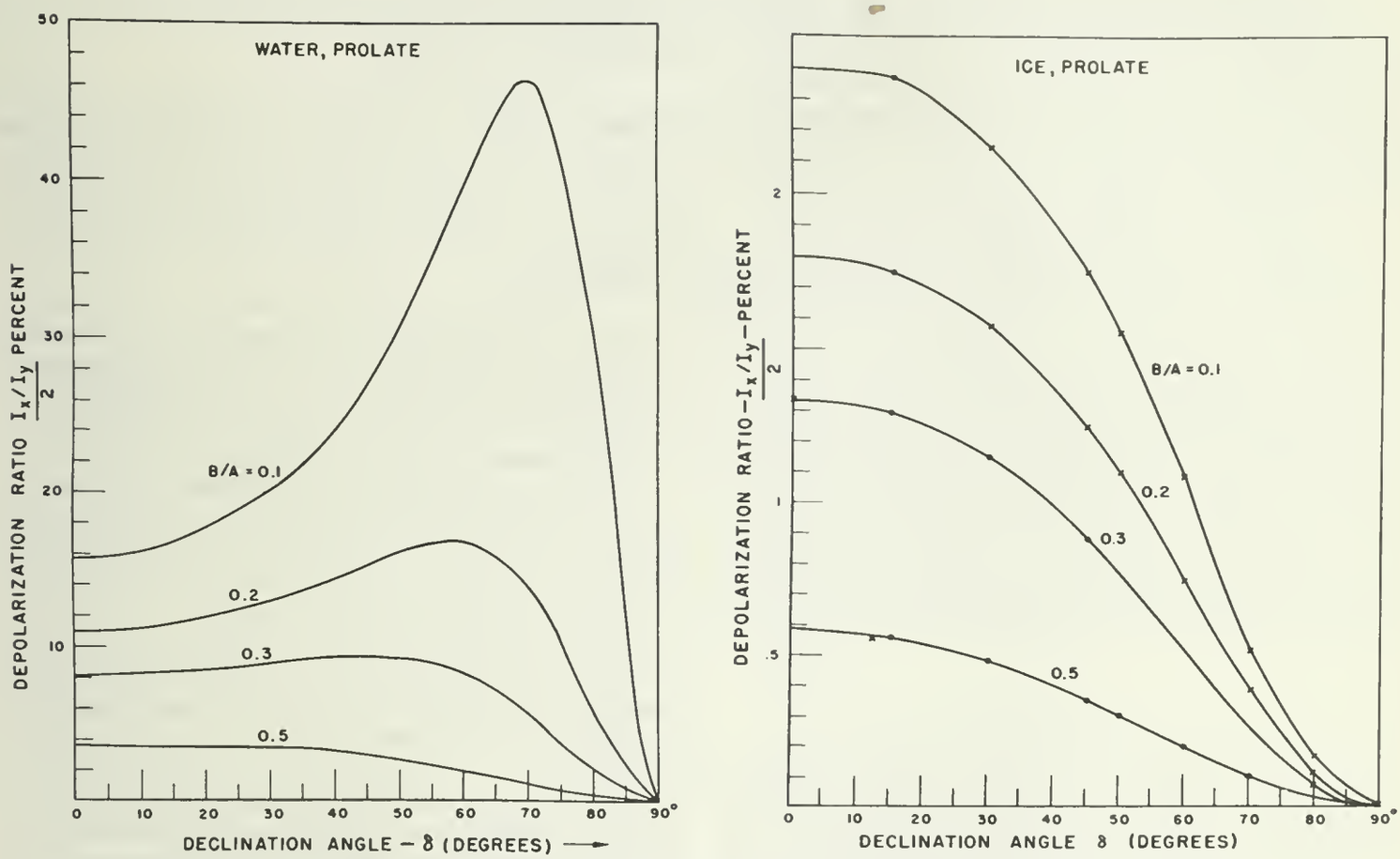

FIG. 131. —THE VARIATION OF THE DEPOLARIZATION RATIO WITH DECLINATION ANGLE FOR HORIZONTALLY ORIENTED PROLATE SPHEROIDS. LEFT, WATER AT 10 CM.; RIGHT, ICE 1 - 10 CM.

should be doubled to get the true values. Again, notice the difference in magnitude of the scales; water goes up to 50% while ice goes only to 2.5 on the half scales. The difference in the two sets of curves is striking. It is due only to the difference in the relative magnitudes of the components for the two cases. The strong maximum for the case of water needles of axial ratio 0.1 at a declination angle of about 70° would make such particles easily identifiable if they were to occur in the atmosphere. Although we should not expect such water particles to occur, very much the same effect should be observed with melting ice needles. The curves for ice should be similarly useful if the small cross-polarized components, up to about 5%, can be detected.

THE BRIGHT BAND

Throughout this talk, I have hinted at the scattering characteristics to be observed with melting particles. Indeed, the only reason that we have carried out the computations for the case of water ellipsoids is that we believe them to be applicable to melting particles and, therefore, to the theory of the bright band. Kerker, Langleben, and Gunn at McGill University have shown that an ice sphere with only a thin coating of water scatters almost equally to a water drop of the same size, even at wave lengths as long as 10 cm. This should also be true for nonspherical particles. In this case, therefore, shape and orientation should play an important part in determining the nature of the bright band.

Consider, for example, that the particles above the melting zone are low density snowflakes, having the form of oblate ellipsoids of axial ratio 0.1, and oriented horizontally. As low density snow, they scatter like spheres regardless of shape. However, as they fall through the melting zone and take on a water coating before changing shape, their reflectivity changes markedly, and also becomes a sensitive function of the elevation angle of the vertically polarized antenna. When viewed from below, there is an increase in reflectivity of about 5 due to index of refraction and about 18 due to shape, giving an increase of 90 to 1. When melting is completed, the sudden collapse to spheres drops the reflectivity by 18 and the increase in velocity may contribute drop of about 4 or 5, thus decreasing the signal

by between 70 and 90 to 1. When viewed from the side with vertical polarization, the reflectivity of the snowflakes may actually decrease slightly in going into the melting zone, there being an increase of 5 due to index of refraction and a decrease of 6 due to shape, or a decrease of 5/6ths. On the completion of melting, shape increases the reflectivity by about 6, and velocity decreases it by 4 or 5.

Of course, we have considered a rather extreme case. With more spherical particles and somewhat less perfect orientation, the effects would not be quite so extreme. The fact that the results are so sensitive to direction of polarization and elevation angle provides us with an interesting experiment for the determination of just how important these effects are. However, we already have some evidence of this. Bright band observations made at vertical incidence generally appear to be more striking than those made obliquely. I believe Dr. Swingle has observed the bright band on a 10-cm. radar at vertical incidence when there were no echoes present below it.

SUMMARY

I should like to conclude my remarks with a brief summary of these results, which, I want to emphasize, are based entirely on Gans' theory for particles small relative to the wave length.

1. First of all, I believe that the general form and state or orientation of the simpler types of nonspherical ice particles may be determined, at least in a qualitative manner, from the characteristics of the back-scattered radiation. There are some ambiguities, but these may be resolved largely from other characteristics of the echoes and the meteorological conditions. This is not the case for low density snowflakes.

2. In general, particles whose axes are oriented obliquely to the plane or polarization of the incident wave will return a cross-polarized component; if the plane of polarization is transverse to one axis of the particle, there will be no such component.

3. The back-scattered radiation from oriented particles will generally be sensitive to variations in both angle of incidence to the plane of preferred orientation and the direction of polarization of the incident wave. An array of randomly oriented particles displays no such sensitivity.

4. This sensitivity is much greater for water-coated particles than for ice of the same shape and size; for ice, it is independent of wave length; for water, the sensitivity increases slightly with wave length over the 1 to 10 cm. range.

5. The radiation scattered from nonspherical particles may be more or less intense than that from spherical particles of the same volume depending on the state of orientation relative to the plane of polarization. Randomly oriented particles always scatter more intensely than equal spheres; oriented particles scatter more intensely when the direction of polarization is parallel to the major axes, and less intensely when parallel to the minor axes. Low density snowflakes scatter very much like spheres regardless of shape and orientation.

6. It is believed that these effects can not help but play an important part in determining the nature of the bright band.

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THE EFFECT OF ATTENUATION ON THE RANGE PERFORMANCE OF RADAR SET AN/CPS-9

BY DONALD M. SWINGLE *

WITH DISCUSSIONS BY P. M. AUSTIN, M. H. LIGDA, F. C. WHITE, G. E. STOUT, E. F. HILL, D. M. SWINGLE

Gentlemen, the title of this paper is perhaps not precisely accurate, since I intend to discuss the considerations which dictated the choice of frequency made in the development of Radar Set AN/CPS-9 and the philosophy upon which that development was based, in addition to the more limited subject of the anticipated effects of attenuation on the range performance of that equipment. However, the title comes close enough, since the relations between the severity of signal attenuation by precipitation, the frequency of occurrence and areal extent affected by such attenuation were predominant in the choice of radar frequency.

Before discussing the choice of radar frequency, I should like to outline the events leading up to the development of the AN/CPS-9 and the philosophy upon which that development was based. From this vantage point we shall be able to view the effects of attenuation in the light of what the radar was designed to do and of what it was not designed to do.

As is well-known to most of the members of this conference, the basic theory describing the scattering of electromagnetic waves by dielectric spheres was derived by G. Mie in 1908 in connection with the problem of the scattering of light by colloidal suspensions. The theory was expanded by several others, applied to the calculation of the attenuation of meter-length waves by J. A. Stratton in 1930, and appeared in his 1941 textbook on Electromagnetic Theory.

It was thus no great shock to the physicists and engineers working with centimeter wave equipment at our M. I. T. Radiation Laboratory and in other advanced development laboratories, when it was found that these waves were effectively scattered by precipitation particles.

When equipment in the S-band (approximately 10-cm. wave length) became operational (approximately at the time of the Pearl Harbor disaster), precipitation echoes were observed to occur with considerable frequency. However, although the echoes were detectable, the portion of the incident radar energy absorbed and scattered by precipitation represented too small an attenuation to be disturbing. It was recognized that these precipitation echoes were the result of Rayleigh scattering and further that both scattering and attenuation would become more pronounced as one progressed toward shorter wave length equipment. It was also recognized that precipitation echo data might conceivably be of some use and/or interest to the meteorologist.

When X-band (approximately 3 cm.) equipment became available, it was apparent from the detailed calculations of Ryde, based on the Mie Theory, that both effects would be troublesome under certain conditions since they would interfere in the accomplishment of the primary purpose of the equipment. However, the other advantages of the equipment outweighed the occasional problems of interference with normal operation of the equipment caused by precipitation.

During the year 1942, Mr. Arthur E. Bent of the M. I. T. Radiation Laboratory studied radar echoes from rain and cloud areas. In 1943, Major J. O. Fletcher worked at the Radiation Laboratory under the auspices of the Army Air Forces Weather Service to determine the interrelations of radar and weather, and, as a result of their studies, a weather radar program was set up by the Weather Service. A number of fields were found to be of interest, but we will consider today only the field of storm detection. The Canadian Army Operational Research Group also entered in the field of storm detection during 1943.

Under the guidance of Major J. O. Fletcher, a group in the Weather Equipment Methods Section of the U. S. Army Signal Corps Ground Signal Agency obtained, set up, and tested Radar Sets SCR-584, SCR-717B and AN/APQ-13 for possible application to weather observation.

As these tests progressed, during the period from 1943 to 1945, there developed a realization that we had in our hands a technique of weather observations which was almost incomparable with any of the previously used techniques, a technique which could be of tremendous value in forecasting and observing the weather.

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If one investigates the history of meteorological observation and instrumentation, one finds that, although the weather has been of great interest to men through all ages, the early observations consisted almost exclusively of qualitative observations. These date back at least to the Babylonians (about 900 B.C.). During the succeeding twenty-four centuries, a fairly extensive weather lore was developed and some of the weather observations were systematized, but by and large little of importance occurred in the development of meteorological instruments until the invention of the mercurial barometer by Evangelista Torricelli in 1643. The barometer remains today the only meteorological instrument which gives data which can be considered to be representative of the surrounding air mass. From 1650 to 1850 meteorological instruments were developed in rapid succession. These instruments were generally intended to determine the weather conditions existing at a given point of observation. This instrumentation and observation system has become the basis for our present synoptic weather network and the surface weather chart.

With the use of data from such instruments, one has attempted to forecast the weather in terms of the fluctuations in temperature, pressure, humidity, wind, cloud cover, precipitation, etc. between a large number of surface stations. By the use of a large number of independent observations, one attempted to obtain a picture of the real continuous distribution of these parameters over the surface of the map, and to forecast their changes. This moderately successful venture has had enough support to establish: first, the Army Signal Corps Weather Network in 1870, and later the U.S. Weather Bureau in 1890, and to keep it going.

The next major advance in instrumentation occurred with the development of radio-sonde during the thirties. Now for the first time, one was enabled to determine the conditions not simply at a point on the surface of the earth, but over a more or less sloping line extending through the atmosphere above that station. An extensive technique of single-station forecasting was developed based on the observations obtainable from such ascents. Still, one was not satisfied with the available data as a picture of the weather, and it was found desirable to establish fairly extensive networks of upper-air stations and also to transmit this data so that an areal analysis might be made.

With the advent of microwave radar, we have been able to progress yet another step in the direction of knowing the state of the whole atmosphere, an essential if one is finally to conquer the forecasting problem by way of a generalized Laplace-problem approach. We are now enabled to determine the conditions with respect to microwave scattering by precipitation not at a point, not along a line above that point, but (a) essentially instantaneously we can determine that distribution over a plane or conical surface about the radar station; (b) given a few minutes, we can obtain the data on other conical surfaces and thus essentially cover the solid volume surrounding the station. Using the simple PPI observation, we have extended the spatial dimensionality of our observation from a line to a surface whose diameter may be from 50 to 200 miles. When suitable three-dimensional presentations are developed, we shall have conquered the problem of observing the distribution of at least one meteorological element throughout a volume. It is in consideration of these factors that I referred above to microwave radar storm detection as a weather observation technique wholly incomparable to those previously existing.

Until the summer of 1945, the only radar equipment in much use for storm detection was the SCR-584, which was also used for the determination of winds. From middle 1944 through the summer of 1945 various light-weight radars were investigated and tested by personnel of the Signal Corps Laboratories.

It was decided, during the early part of 1945, to adopt the AN/APQ-13, a standard airborne radar normally used for navigation and bombing, as an interim storm detection set pending design of a special storm detector by the Signal Corps Engineering Laboratories. A special Weather Radar Section was set up at Evans Signal Laboratory with Mr. Gould as Chief. Among its first tasks was the development of the AN/CPS-9. The fruit of this development was reported by L. A. Zurcher of the Evans Signal Laboratory at the Washington, D. C. meeting of the American Meteorological Society in the Spring of 1949. Mr. Zurcher emphasized, as I wish also to do, that the AN/CPS-9 has been designed specifically for use by the meteorologist as an operating tool in weather forecasting. Although it offers many possibilities for research in the fields of precipitation physics and general synoptic meteorology, all factors entering into its design and development were considered in terms of their effects on the day-to-day operation and use of the equipment as a storm detector by the Air Weather Service.

Because of the manner in which microwave transmitting equipment is developed, the choice of operating frequency was by all odds the most difficult engineering decision to be made in the development of the equipment. It was not extremely difficult to establish limits on the resolution and accuracy required of the equipment, thus setting the values of the pulse length, receiver band width, and the ratio of antenna aperture to the radar wave length. One was then left to suitably adjust the frequency to optimize the performance of the equipment, while still remaining within reasonable size,

weight and power limitations. This would have been a difficult job in itself. However, one had also to consider the availability of transmitting magnetrons and other components at each particular frequency. This served to reduce the problem from that of determining the optimum frequency from theoretical calculations only, to one of determining which of a set of possible frequencies would do the best job.

One can perhaps best visualize the problem by consideration of the so-called basic storm detection radar equation in which the presence of frequency dependent factors is indicated. We consider the form where it is assumed that the radar beam is completely intercepted by area of uniform precipitation.

$$\left(\frac{S}{N}\right)^2 = \frac{C\,\Sigma}{R^2 i}\; n_i a_i^6\; \frac{1}{\lambda^2}\; \frac{D}{\lambda}^{2}\; \frac{1}{n(\lambda)}\; \delta^2(\lambda)$$

$$P_t(\lambda)\; \exp\left[-2\int_0^R K(\lambda, r)\, dR\right]$$

$\left(\frac{S}{N}\right)^2$ = the ratio of signal to noise power at the receiver.

C = a frequency independent constant.

R = the range from the radar to the precipitation region in question.

$\Sigma_i n_i a_i^6$ = summation of sixth powers all droplets contained in unit volume in the precipitation area.

D = diameter of paraboloidal transmitting-receiving antenna.

n = noise figure of the receiver.

δ = duration of transmitted pulse.

P_t = peak transmitted power.

K = attenuation coefficient per unit depth for rainfall of rate r.

Here one notes that the ratio of signal to noise power received from a given precipitation region is a function of:

1. The ratio of the diameter of the antenna system to the radar wave length, limits on permissible values of this ratio being given by resolution requirements. Further limitations on the diameter of the antenna system are dictated by considerations of wind loading, weight, and power requirements.

2. The noise figure available at any given wave length, a function of the wave length and available components.

3. The pulse duration available at any given wave length, depending primarily on the availability of magnetrons designed to produce it. This appears in the square because of the influence of pulse duration on receiver band width. This also affects the weight, size and power consumption of the modulator.

4. The peak power available at a given wave length, which also depends on the development of suitable magnetrons. Choice of power will have a strong effect on the weight, size and power consumption of the modulator.

5. The attenuation to be expected from any given precipitation situation, which depends on the wave length and could be critical.

The design problem was to arrive at the best possible compromise of these various parameters involving the radar wave length, while yet making the equipment as useful as possible. The equipment was not intended to be a research tool, nor was it possible then (as it is not yet possible) to use the radar as an accurate measure of the intensity of precipitation. This is one of the problems concerning this conference and remains one of the major problems in the radar storm detection field. The AN/CPS-9 has instead been designed within the restrictions outlined above, to detect precipitation to as long ranges as possible whenever such precipitation occurs above the radar horizon. The design has been quite successful in this respect, obtaining echoes more frequently and to greater ranges than is possible with any other radar equipment of comparable weight, size and power consumption. Thus it takes the maximum advantage of the unique capability of microwave radar to detect precipitation and to represent the general spatial distribution of that precipitation over a large area.

The manner in which the operating wave length of AN/CPS-9 was chosen is outlined below. As soon as storm detection was shown to be a worthwhile venture, the group at the Weather Equipment Methods Section of the Signal Corps Laboratories began an active theoretical and experimental investigation of the factors leading to the optimum performance of storm detection radar. As a result of this study the AN/APQ-13 radar was adopted as an interim storm detection set, as indicated above, and an active project for the development of the AN/CPS-9 equipment was established. The investigations, particularly of the effect of attenuation, were continued by the Signal Corps Engineering Laboratories.

In 1947, Wexler and Swingle published an abbreviated theory of radar storm detection for the consumption of the meteorologist. Several of the results of the Signal Corps' studies were included in this article, the most interesting being the effect of attenuation on the detection of precipitation of a given intensity over a path where precipitation of the same intensity was falling. For the complete interception case, it was shown that a rainfall of approximately one-half inch per hour would give a greater radar echo at 3.2 centimeters than a fall of 0.2 inches per hour to a range of approximately

10 km., subsequently producing less signal because of the greater attenuation. Similarly, again at 3.2 cm. wave length moderate rain would be detected through continuous moderate rain of 0.2 inches per hour with a greater signal level than light rain through light rain of .04 inches per hour to a range of 75 kilometers. It is therefore evident that a radar operating in the 3 cm. range will be subject to attenuation, which might at times become severe.

At the same time that these attenuation studies were in progress, the engineering factors of size, weight, power consumption, availability of components, etc. were being evaluated. It was apparent that any S-band radar of reasonable size, weight, power consumption and cost could not begin to compare with the performance of an X-band equipment having similar gross characteristics, nor would such an S-band equipment take full advantage of the potentialities of the storm detection technique for the observation of weather over an area.

Realizing the possible importance of attenuation as affecting the performance of the equipment, studies were made along two lines. The first was to determine the frequency and severity of attenuation likely to be experienced by various possible designs. The second was to determine the availability of components in the region between 11 and 2 cm. wave length, since it was felt that perhaps a wave length in the range from 5 to 7 cm. might be the optimum.

The results of the second study were almost completely negative. Except near λ = 3.2 cm. and λ = 10 cm., no components of reasonable characteristics had been developed, nor were any under development or approaching production. To enter into a development appeared to be unwise, since that would hold up all work on the equipment for two to four years.

Thus the wave length choice was narrowed from the range of 2 to 11 cm. to a choice between approximately 3.2 and 10 cm. There was no possibility of compromise to middle ground.

Concurrently, the second study mentioned above, an investigation of the frequency and extent of attenuation of 3.2-centimeter waves by rainfall was carried out, the major results being reported by Wexler and Weinstein (2). It was recognized that heavy rain may occasionally block the radar from detecting storms beyond 25 miles.

Using the Robertson and King empirical X-band attenuation value (0.03 db/km. per mm. per hour rainfall), which is 50% greater than that calculated by Ryde, curves relating rainfall rate, radar range and attenuation were prepared assuming radars approximately equivalent to the AN/APQ-13 and AN/CPS-9. Under the assumption that the precipitation region always fills the beam, the maximum range of this equipment on rain of light (1.25 mm. per hour) and heavy (10 mm. per hour) intensity, with no intervening attenuation and with attenuation due to uniform rain of various intensities are given in Table I.

Since these figures indicate that 3.2 cm. radars, even those of great sensitivity, may at times encounter severe attenuation effects, Wexler and Weinstein conducted three further investigations summarized below.

Considering first the frequency of hourly rainfall amounts at Boston (11 years), Columbus (12 years), New Orleans (30 years) and Oklahoma City (25 years), they found that 60 to 80% of all measurable summer rains were in the light category (less than 0.10 inches per hour), while 80 to 90% of all winter rains were in this category.

Intensities of 0.40 inches per hour (10 mm. per hour) or greater ranged from 2.4% to 11.2% (3 hours to 13 hours) of the summer rain hours at the stations. The pessimistic attenuation figures for such rains are shown in the table below as they affect the AN/APQ-13 and AN/CPS-9 radars.

Since most heavy rains are of short duration, it appeared likely that they are not widespread. For instance, in Boston, during the 10-year record period of summer rains, only 35 storms occurred

Table I

Maximum Range to Which Light and Heavy Rain Can Be Detected Through Intervening Rain of Various Intensities

	Light Rain (1.25 mm/hr.)				Heavy Rain (10 mm/hr.)			
	0	1.25 mm/hr.	5 mm/hr.	10 mm/hr.	0	1.25 mm/hr.	5 mm/hr.	10 mm/hr.
AN/APQ-13	80 km.	60 km.	30 km.	22 km.	240 km.	120 km.	55 km.	35 km.
AN/CPS-9	600 km.	230 km.	90 km.	56 km.	1800 km.	320 km.	120 km.	75 km.

which had at least one mean hourly intensity of 0.3 inches per hour or more (heavy rain) while four of these showed two consecutive hours of heavy rain and only one showed three consecutive hours of heavy rain.

From this study they concluded that heavy rains occur infrequently and that heavy rains covering large areas are rare.

A second study was made to determine the areal extent of heavy rainfall. Using a network of four stations 13 to 15 miles apart near Houston, Texas, they analyzed the hourly rainfall data for September for the years 1942-1945, that month, in the hurricane season, being chosen as one in which widespread heavy rain was most likely to occur.

Over the six-year record period, at least one station reported heavy rain (0.3 inches per hour or more) for a total of 54 hours. During only one hour of this period did heavy rain occur at all four stations simultaneously, while simultaneous heavy rain occurred for four hours at three stations, nine hours at two stations, and forty hours at only one station. By contrast, there were twelve hours during which heavy rain occurred at one station and no rain fell at any of the other three stations. Wexler and Weinstein concluded that the odds were better than 3 to 1 that heavy rain will not extend to a diameter of 25 miles or more.

As a final study, the number of hours per year during which the AN/CPS-9 would be reduced in range below 50 miles in the detection of heavy (0.4 inches per hour) rain were investigated. Considering the half-hourly maps of the Muskingum Valley watershed for the year 1938, they found that the mean precipitation intensity was 0.2 inches per half hour from the center to a point 50 miles in any direction for a total of 20 hours, of which 19 corresponded to line squalls or lines of thunderstorms, the remaining rain being due to a single large (25-mile diameter) thunderstorm having extremely heavy rainfall. Only one hour occurred in which a squall line passed over the station, causing the specified attenuation along lines on both sides of the station. In no case was the intensity of rain 0.20 inches or more per half hour in all directions.

From these studies we concluded that the performance of X-band radar for storm detection would not, in most locations, be seriously limited by attenuation due to rainfall.

It is indeed fortunate that this is so, for consideration of the design of a radar having equal abilities, but not subject to severe attenuation operating at a 10 cm. wave length shows that it could not be made within reasonable size, weight and power limits. For instance, to retain good resolution we would require an antenna scaled up to about 25 feet in diameter. This antenna would have to be much stronger and heavier per unit area to withstand the increased ice and wind loading, while the pedestal and drive system would have to be similarly scaled up and strengthened. We would still need to scale up the transmitted power by a factor of ten, resulting in an appreciably heavier and larger modulator. Both these changes would act to increase the over-all power consumption and, perhaps more important, to increase the cost of the equipment. But since the AN/CPS-9 already pushes the size and weight limits set by the requesting Using Service, such a 10 cm. radar could not have been acceptable and we would have no AN/CPS-9 today.

As was noted already, the delay in waiting for or initiating development of a whole new line of components in 5 to 7 cm. range was regarded as intolerable.

Thus we were forced by circumstance to accept 3 cm. as the operating wave length or do without the radar, while our background studies of rainfall and attenuation at 3.2 cm. wave length showed that, although attenuation would at times be severe, the usefulness of such a radar as the AN/CPS-9 would not often be seriously impaired by attenuation due to rainfall, in most locations.

We have now had experience with the AN/CPS-9 for something like three years. Attenuation has not proven to be serious at any of the three locations in the East where the radars have been used. It has most certainly not been eliminating echoes from 10% of the total radar coverage area for as much as 10% of the time.

In our New Jersey experience, attenuation has been noticeable, resulting in loss of echo from large areas, in perhaps one or two cases per summer. This has assured us that we made the correct decision, under these circumstances when the radar wave length was frozen at 3.3 centimeters, that is in terms of the intended use of the equipment; i.e., to detect as much of the precipitation area as possible and to do so as often as possible. The equipment is not intended to do a 100% job on a much smaller area, nor to avoid attenuation altogether so as to permit its use as a research tool or to plot contours of echo strength, the latter use still being in the research stage.

Should it appear, in the course of time and research, that the field equipment would be better used by changes in its design, these could be effected either by the addition of a modification kit or by production of a second model incorporating such changes as are felt necessary or desirable. At such time it will be up to the using services to state their newer requirements.

At the present time, however, the set appears to be the best that could be had by way of obtaining good sensitivity and good coverage most of the time, with very occasional difficulties resulting from rainfall attenuation.

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DISCUSSION

P. M. AUSTIN. —Is there any way of using the attenuation effect? You have a good radar-weather set, and if we can use it to actually measure the attenuation, it might be well to leave the attenuation in, thus helping to determine precipitation intensity.

D. M. SWINGLE. —That is a possibility which deserves further investigation. Ray Wexler did make an effort in this direction in his paper on "Radar Detection of a Frontal Storm."[1]

UNIDENTIFIED. —What is the longest range you have obtained?

W. B. GOULD. --I believe we have had actual precipitation detected to something like 350 miles.

D. ATLAS. —I still think that you have to find a purpose, and I believe the greatest utility of the radar set will be in somehow giving us a synoptic picture.

M. H. LIGDA. —I think that Mr. Gould's outfit really rang the bell with the CPS-9, but having the CPS-9 and the SCR-615B (3- and 10-centimeter systems) side by side at M.I.T., we have been struck by the difference in presentation and in many cases by the similarity in presentation which are obtained from these radars. The SCR-615B has a three-degree beam width that tends to broaden small precipitation cells detected at a given range; the better performance of the CPS-9 results in the presentation of a cell of about the same size because, although it has a one-degree beam width, it is seeing the lighter rain which cannot be detected by the SCR-615B. One thing that we have been troubled with (which is going to be an operational problem) is that the CPS-9 sees too much rain! It is going to present a considerable problem in the interpretation of the synoptic picture in that you are going to find a good many operators running a set with the gain settings so high as to wipe out important features of the heavy rainfall with the extraneous detail from the weak rainfall. The CPS-9 performs beautifully by virtue of its resolving power and short wave length, but sometimes it just sees too much, and you are just confused by the picture. The ten-centimeter equipment could have been (and has been) considerably improved, keeping in mind that the SCR-615 was designed perhaps back in 1941 or 1942 and nobody has really put very much more work in on it since. It has roughly three times the power output of the CPS-9, and this could have been increased appreciably. I would like to say that we are going to be troubled by seeing this very light rain at close range; it is going to complicate things. Often you are not going to know whether it is rain or cloud that is being detected.

D. M. SWINGLE. —The CPS-9 is presumed to be run by intelligent operators. It is provided with both gain and sensitivity-time-controls. This would take out the light rain if it bothers one.

F. C. WHITE. —I would like to add an amen to Mr. Jorgensen's comment there with regard to the other uses of radar, speaking for one of the organizations that would like to use radar. I personally fly with Lt. Montgomery here. Once we flew 12 minutes in a thunderstorm at 140 knots when the radar told me that it ended 3 miles to the other side of us. Personally, I do not want to do it any more.

G. E. STOUT.[2]--This summer we had an area where there was heavy rainfall (greater than 1 inch per hour) for 50 miles' depth, and all the 3 cm. radar (APS-15) showed was the first 10 miles of it. Within a 200-mile radius these heavy storms will occur about eight or ten times a summer in the midwest area.

E. F. HILL.[3]—I would just like to say that we have an excellent collection of high-powered components on the slightly less than six-centimeter band.

[1]R. Wexler, "Radar Detection of a Frontal Storm, June 18, 1946," J. Meteorology, pp. 38-44, Feb. 1947.

[2]Meteorologist, State Water Survey, Urbana, Ill.

[3]Electronics Project Engineer, Glenn L. Martin Co., Baltimore, Maryland.

THEORY OF THE RADAR UPPER BAND

BY RAYMOND WEXLER[1]

WITH DISCUSSIONS BY D. C. BLANCHARD, M. H. LIGDA,
J. S. MARSHALL, R. WEXLER

(Summary)*

The observations of the radar upper band are briefly reviewed. The growth of graupel in a supercooled water cloud is analyzed. There exists a critical value of the liquid water content below which crystal growth by diffusion predominated and above which the crystal converts into graupel of quasi-spherical shape. The growth of a layer of graupel and the subsequent depletion of the cloud liquid water content below the critical value is capable of explaining the observations pertaining to the upper band.

[1]Research Meteorologist, Mt. Washington Observatory, 102 Mt. Auburn St., Cambridge, Massachusetts.

*The complete article will be published in the Quarterly Journal of the Royal Meteorological Society.

DISCUSSION

D. C. BLANCHARD.*—Can you give more information concerning the collection efficiency that you used in your calculations?

R. WEXLER.—The collection efficiencies for sphere, cylinders and ribbons have been computed by Langmuir and Blodgett. The collection efficiency for a circular disc, to which a snow crystal can be approximated, may be deduced by comparing values for sphere, cylinder and ribbon. For viscous flow it is estimated that a crystal of radius 75 microns and thickness 15 microns has a collection efficiency of more than 50% on droplets of 10-micron radius. The collection efficiency of dendritic crystals of radius approaching 1 mm. is much less.

*Meteorologist, Woods Hole Oceanographic Ins.; Woods Hole, Massachusetts.

M. H. LIGDA.—Can you give a little more detail as to how you concluded that the upper band was not due to wind shear?

R. WEXLER.--Bowen's observations were made with a PPI which scanned from horizon to horizon along the vertical. The upper band certainly looked horizontal in his motion pictures of the phenomenon. It was estimated by Dr. Marshall and myself that a wind shear of more than 150 miles per hour in a height interval of 1 km. would be required to give the appearance of a horizontal band.

J. S. MARSHALL.—This paper shows what happens when a perfectly innocent weather-radar researcher studies cloud physics in England for two years. He returns talking about dendritic growth, splintering and chain reactions!

AIRBORNE RAINDROP SIZE MEASUREMENT AND INSTRUMENTAL TECHNIQUES

BY ROBERT M. CUNNINGHAM*

WITH DISCUSSIONS BY J. S. MARSHALL, D. C. BLANCHARD, R. M. CUNNINGHAM

This paper will consist of a quick review of our attempts over a few years to build an optical raindrop measuring instrument which, as a great many of you know, is called a disdrometer (drop-size distribution meter). The paper also contains some observations below a "bright band" taken with the aid of this instrument.

The problem of sampling a sufficient number of drops with the disdrometer is discussed.

The inadequate sample taken by this instrument has prompted us to experiment with a different type of head unit. A brief description of this new drop-measuring device is included at the end of this paper.

Fig. 132 is the cross-section of the optical disdrometer. Very briefly there is shown the light

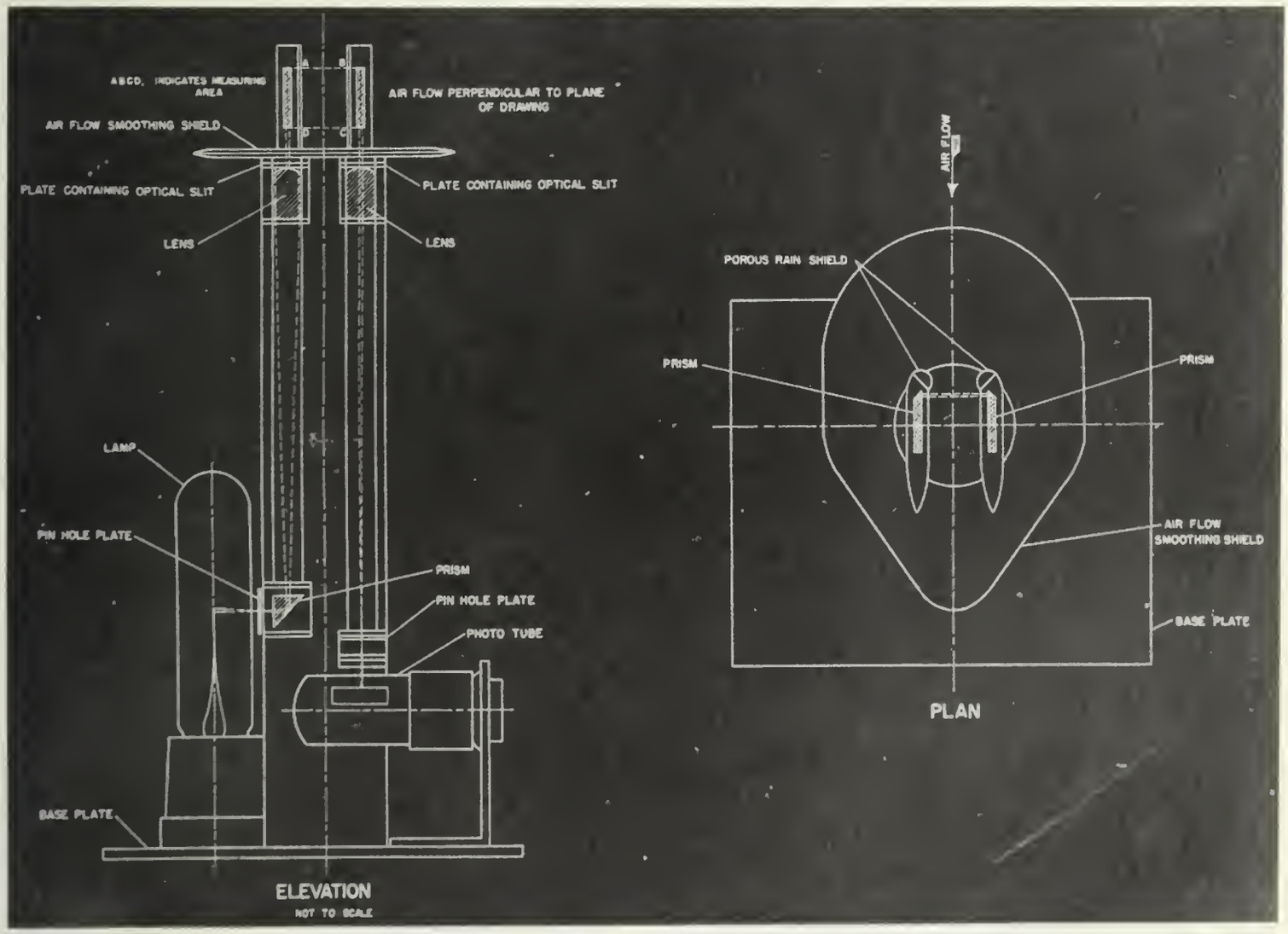

FIG. 132.—CROSS-SECTION OF OPTICAL DISDROMETER.

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source, lenses and prisms. The parallel light beam then travels across the two arms up at the top. The motion of the air and the motion of the drops is perpendicular to the plane of the screen. The second prism sends the light beam back down to a phototube. A drop going through the beam between the two posts casts a shadow and reduces the amount of light arriving at the phototube. The amount of reduction in the light is proportional to the projected area of the drop onto the receiving slits at any instant of time. The magnitude of the resulting negative "pip" on the D. C. output level of the photomultiplier tube is then also proportional to this shadow area, or indirectly to the drop size. This "pip" is then sent through electronic circuits and eventually to a six-channel counter.

There were many extraneous problems that had to be solved (besides the major electronic one of getting a complicated counting circuit to work reliably in an aircraft). These problems largely involved the head unit. The prisms had to be kept dry while heavy rain passed by, one mm. away. The front edges of the posts consisted of porous metal. A vacuum system supplied 6 to 10 inches of vacuum to these posts, which kept them dry. Dry nitrogen was passed around the prisms' surfaces to insure that they did not get wet, either from impingement or condensation. These precautions have kept the optical surfaces dry in very extreme laboratory wind tunnel tests and also in most flight tests in heavy rain.

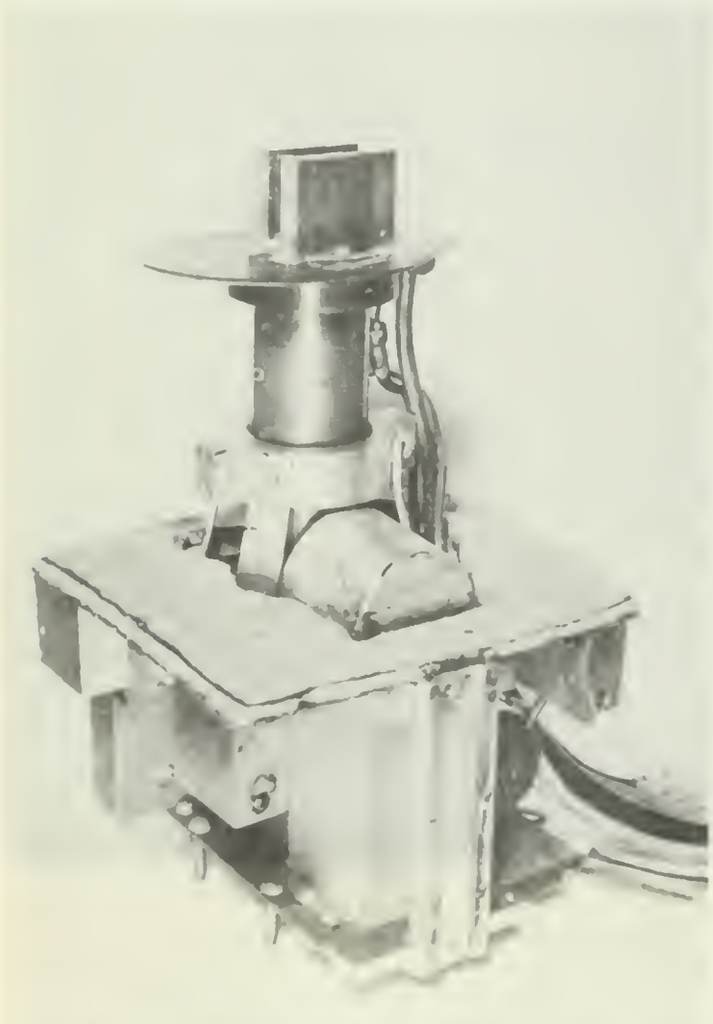

FIG. 133. —OPTICAL DISDROMETER.

Fig. 133 is a photograph of the instrument. The two posts are clearly shown. One slit with the prism in behind is visible. The large plate below the posts is there to keep the air flow as straight as possible. The instrument was mounted out through the former astrodome hole of a B-17. The astrodome was replaced by a special mounting. The disdrometer measuring section was 18 inches above the surface of the nose of the plane.

An example of some data taken with the disdrometer is shown in Fig. 134, which shows some ground radar photograph pictures taken during the flight. To the upper left is the PPI from a 10 cm. radar with an elevation angle of 4°. To the upper right is the PPI from a 3 cm. radar with an elevation angle of 1°. Two representative RHI (3 cm.) pictures are shown at the bottom of the figure. Note the fairly distinct bright band with no echo above and increased echo in the lowest layers, presumably due to drop coalescence. In Fig. 135 we have plotted, on a moving coordinate system, a cross-section of the weather and radar picture. The zero mileage point is moving with the center of one shower shown on the RHI radar. The flight path is plotted as a dashed line. The cross-hatching represents a composite RHI radar echo; double cross-hatching represents relatively stronger echo. No echo occurs below 2,000 feet because of the shadowing by the Boston buildings. The region represented by sparse dotting above 9,000 feet is the region of snow and ice crystals. Light rain fills the volume below 9,000 feet being relatively heavier in the regions of radar echo. The dark gray areas are regions of cloud.

Observations with the disdrometer in the snow region (the instrument gives a rough idea of size and number of snowflakes) check with our idea of the growth of snow near the bright band. Above the bright band (Run I) almost all of the flakes were of the smallest size perceptible. At the top of the bright band (Run II) the total number of drops was two to three times greater than at any other level distributed as in Run V. The number of drops recorded on Run IV at the bottom of the band was in between Runs II and V. Observations in rain with the disdrometer were taken along the flight path marked V, VI, and VII in Fig. 135. Run V sampled the rain just after it had melted from snow, Run VI after the rain had fallen through the upper stratocumulus deck, and Run VII below all cloud decks. Fig. 136 presents the smoothed raindrop size distributions for these three runs. Little change is in evidence between Runs V and VI, but Run VII indicates that considerable growth of the larger drops took place. This conclusion agrees with the radar

picture of Fig. 135. Rough theoretical calculations of growth from coalescence give growth values from Run VI to VII of about one half the magnitude measured.

The vertical temperature structure as measured by a vortex thermometer is shown on the left of Fig. 136. Note the isothermal region at the bright band level. This is the region where most of the smaller ice crystals were melting. A few snowflakes were still observed during Run IV.

Little further analysis of the type just described has been carried out because we have grave doubts as to the reliability and accuracy of the measurements made with the disdrometer. Fig. 137 presents a comparison of the water content as measured by the capillary collector and that calculated from drop size and number data from the disdrometer. At low values of water content the points scatter on both sides of the 1:1 line, but there are a few cases where the disdrometer indicated four times as much water as the capillary collector. At higher liquid water contents the disdrometer indicates as much as thirty times too much water. A survey with a movable capillary collector was made at various heights over the astrodome but no regions of significantly higher water contents than given by the standard location were found. At the moment, I believe that the large acceleration forces in the vicinity of the disdrometer distort and break up enough drops so that a false size is assigned to each drop. To calculate the drop size from the size of each pulse on the electrical counting system it is necessary to assume that the pulse is produced by only one spherical drop.

It will be interesting to find out what results N.R.L. will have with their disdrometer when they fly it, exposed as it is on top of a 15-foot mast.

Aside from difficulties due to our disdrometer's

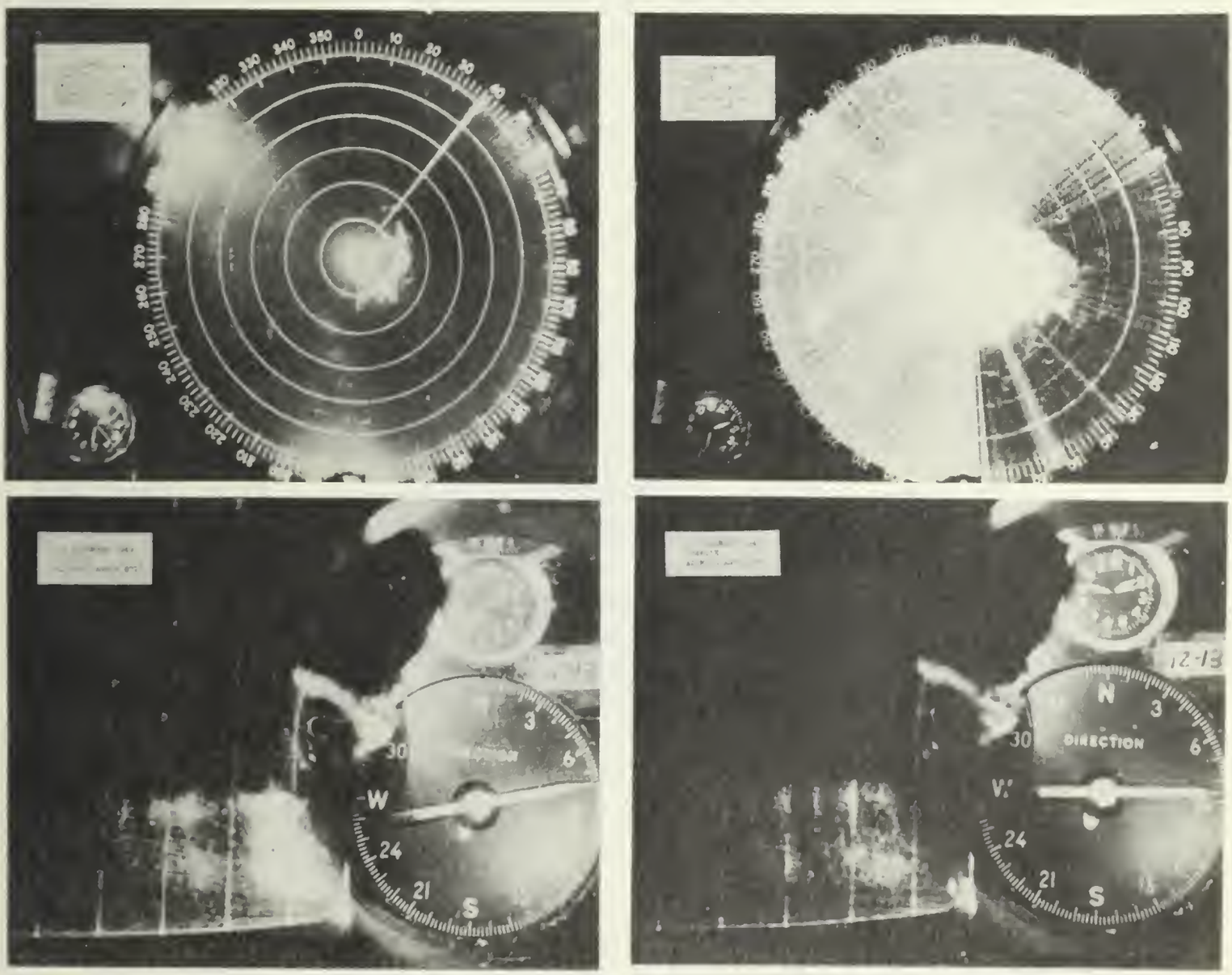

FIG. 134. —GROUND RADAR SCOPE PHOTOGRAPHS.

exposure on the airplane, there is a fundamental difficulty in getting a sufficient sample with the necessary small sampling area. A study was made in order to find out what the optimum sampling area should be. Fig. 138 shows data of Laws and Parsons reworked for our use. The curves sloping from upper left to lower right represent distributions for various water contents. The other crossing lines represent cumulative percent values along the curves of various water contents. For instance, for a rain with a total water content of 6 g/m.^{3} there would be 9 drops per m.^{3} made up of drops of 4.7 mm. in size or larger. These drops would make up the first 10% of the total water content counting from the largest drops down.

Using information from this, Fig. 139 was prepared. This figure shows that a good statistical sample will be obtained for combinations of sampling areas and water contents that fall between the two sloping lines. Note that the present disdrometer sampling area is only adequate at very high water contents.

We can see that there is not any size of sampling area that will be adequate for all possible water contents, although a size of about 100 cm.^{2} will cover most cases. It is obvious that one head unit cannot give good results for all the possible intensities of rain. However, two or three head units could. These facts were realized to some extent when we first designed the disdrometer. It was planned that the present disdrometer would be one of a series which had different sampling areas. The sampling size of the existing disdrometer was at that time largely determined by the engineering requirements. From the experience gained in building and testing the first disdrometer, we can now conclude that it would be extremely difficult to build other heads of larger size to cover the more ordinary rain rates.

We have recently looked for a simpler type of head unit—one which is relatively simple to build. This past spring and summer we have been working

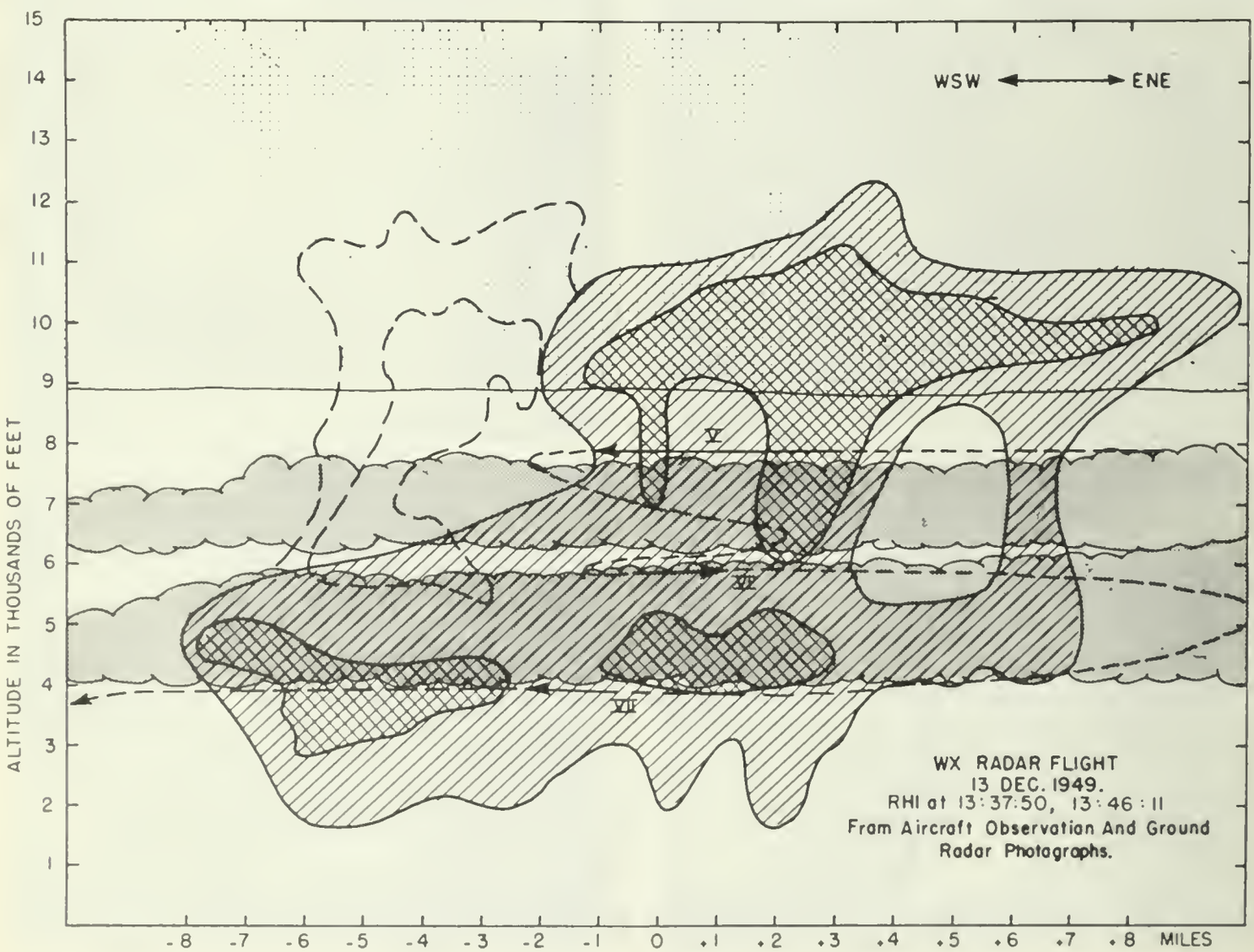

FIG. 135.—CROSS-SECTION OF WEATHER AND RADAR PICTURE.

on an impact or microphone-type head unit. Fig. 140 shows a cross-section of our latest model. There are three principal problems involved in a gadget of this sort, besides the one of calibration.

1. The gadget must be so designed that the noise produced by the impact of a drop will be much larger than aircraft noise.

2. The oscillations produced in the gadget by the impact of a drop must be damped out very quickly so that the head unit can be ready to measure the next drop; that is, the recovery time (measuring time in Fig. 139) should be as short as possible.

3. The response across the sampling area must be constant for the same impact.

A very simple impact device was flown last February. The results of these tests convinced us that the aircraft noise problem is not a major one. Our present model was built in an attempt to solve difficulties 2 and 3. In order to get uniform response over the sampling area, a membrane (2) was stretched across an oil-filled chamber (blackened area). The sampling portion of this membrane was covered with an aluminum plate (5). An inner baffle plate (3) was placed inside the oil chamber. By adjusting the sizes and materials used, a combination was found for one head unit which gave quite uniform response across its sampling area. An attempt at shortening the response time has been made by adding mechanical damping along the connecting rod and by using a voice coil (velocity type) pick-up so that the motion of the coil could be elec-

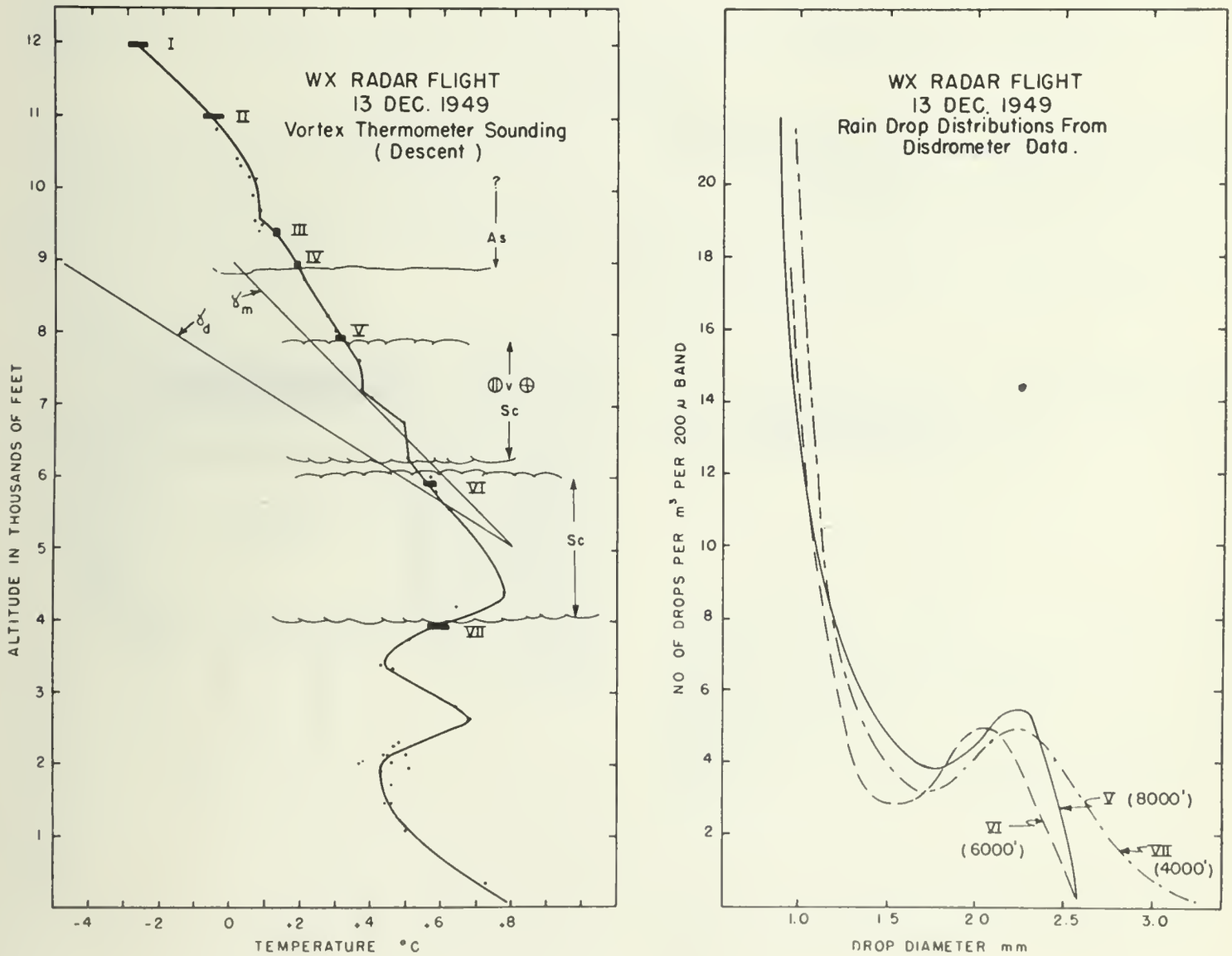

FIG. 136. —SMOOTHED RAIN DROP SIZE DISTRIBUTIONS AND TEMPERATURE DURING THREE RUNS.

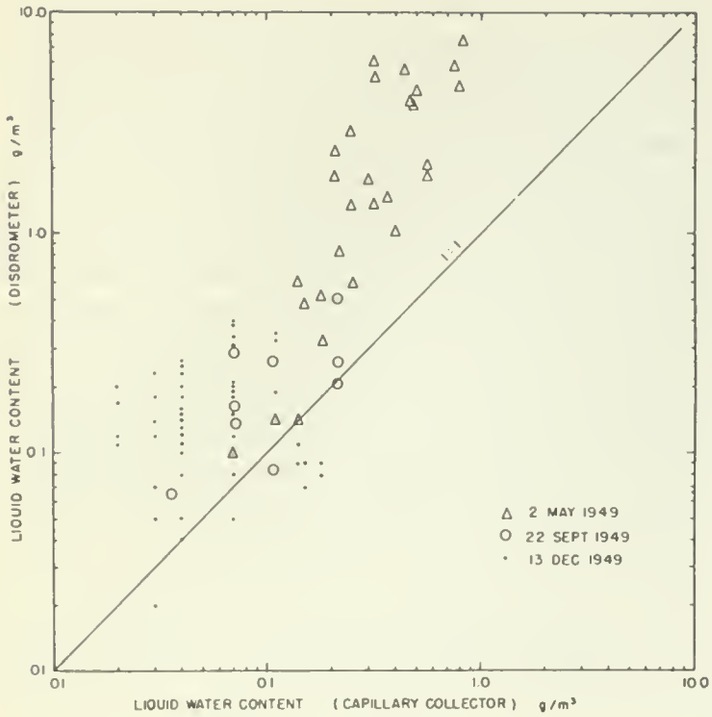

FIG. 137.—COMPARISON OF LIQUID WATER CONTENT AS DETERMINED BY THE DISDROMETER VS. CAPILLARY COLLECTOR.

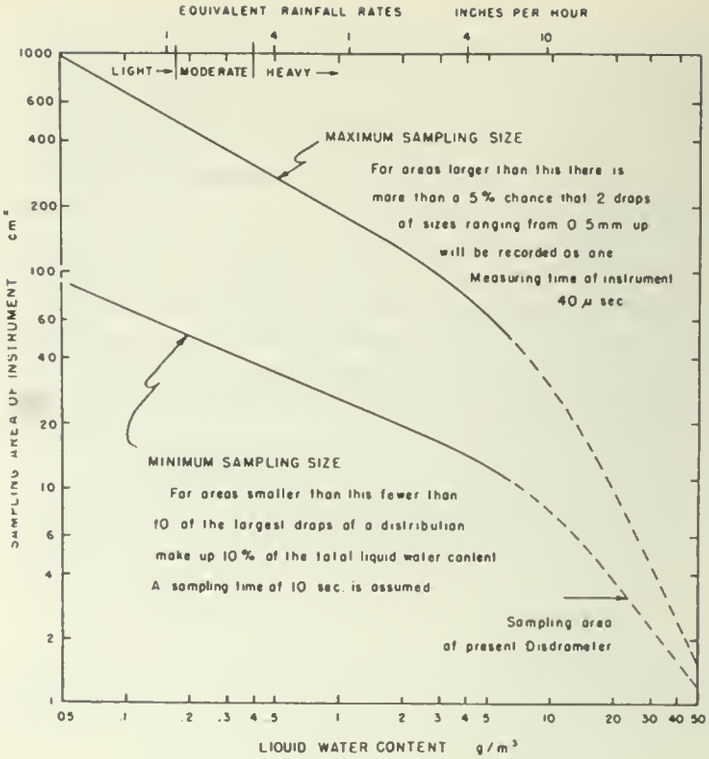

FIG. 139.—EFFECT OF SAMPLING AREA ON STATISTICAL ACCURACY.

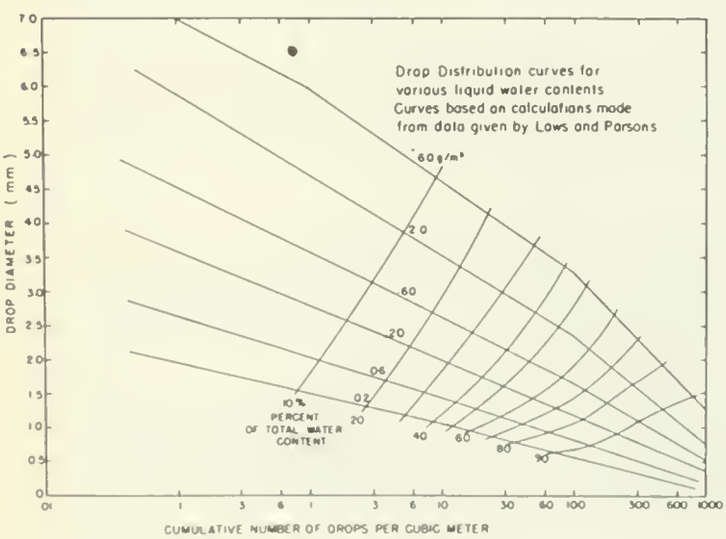

FIG. 138.—DROP DISTRIBUTION CURVES FOR VARIOUS LIQUID WATER CONTENTS.

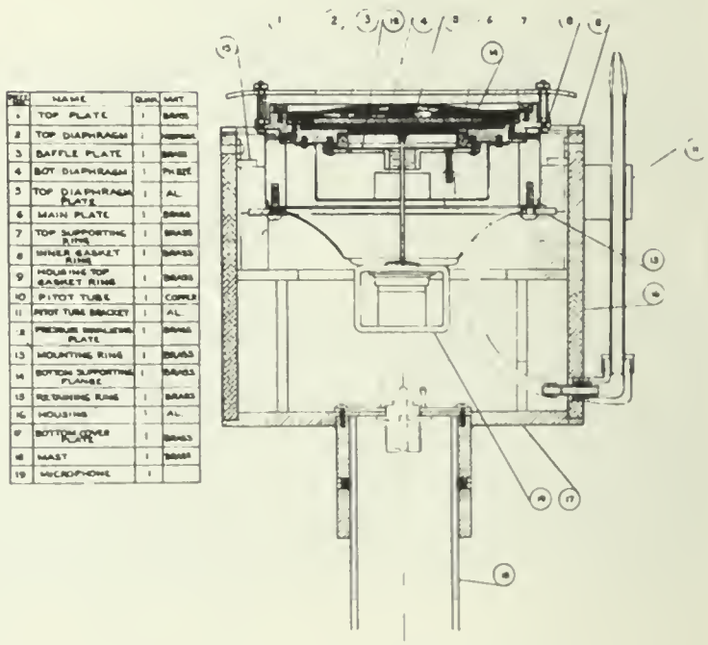

FIG. 140.—CROSS-SECTION OF IMPACTOMETER.

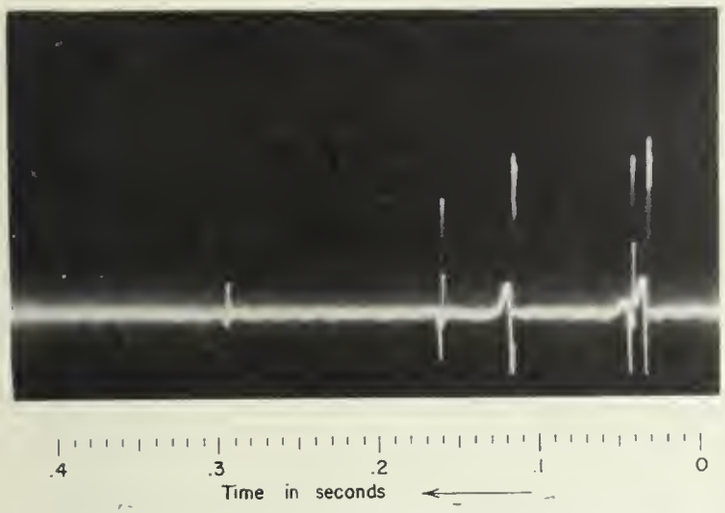

FIG. 141. —EXAMPLE OF IMPACTOMETER CAMERA FILM.

trically damped.

Fig. 141 shows a short length of oscillograph film from the first in-flight trial last week of our present instrument. Note that the original pulse is much larger than the following oscillations but that these oscillations still exist, extending the response time to 10 to 100 milliseconds—much longer than the disdrometer's 40 microseconds. This great difference is not as serious as it first seems because, by using a combination of several head units, each for a limited drop size range, a much larger response time for each unit is permissible.

It is obvious that the work so far on this type of head unit is quite preliminary.

At the present it is planned that considerable laboratory work will be carried out on the impact instrument before further flight tests are made.

DISCUSSION

UNKNOWN. —Just out of curiosity, what are the breaks, in your mind, in the Laws and Parsons distribution curves you showed (Fig. 138). There seem to be very definite breaks.

R. M. CUNNINGHAM. —These probably should have been smoothed out.

J. S. MARSHALL. --You might say they were caused by computational noise!

D. C. BLANCHARD. —Just a remark about your measured drop distribution figure. As I recall, the figure showed the number of drops per cubic meter versus diameter. Is that right? The values shown for the number of drops seem quite small.

R. M. CUNNINGHAM. —I believe that is because the data were plotted using a small band width. A 200-micron band width was used.

RESULTS OF MEASUREMENTS OF RAINDROP SIZE

BY ROLAND J. BOUCHER*

WITH DISCUSSIONS BY D. M. A. JONES, R. WEXLER, R. J. BOUCHER

ABSTRACT

A technique of measuring raindrop size distribution using a nylon screen is described. Results of 63 rain samples obtained by this method at Cambridge give a value of $Z = 269\ R^{1.55}$ by a regression of log Z on log R. The standard error is 47%. A better-fitting line, obtained by trial and error, is $Z = 180\ R^{1.55}$, which reduces the standard error to 38%.

DESCRIPTION OF SAMPLING TECHNIQUE

Previous techniques for measuring raindrop size consist of collecting samples on sheets of absorbent paper on which a water-soluble dye, such as potassium permanganate or methylene blue, has been dusted. Upon striking the paper the raindrops produce permanent stains which can be converted into drop sizes. A second technique is the flour pellet method. This consists of allowing raindrops to fall into shallow pans of freshly sifted flour, forming pellets of dough which can be dried, sorted, weighed and counted to arrive at a drop-size distribution. In using the dye-paper technique it has been found that the splattering of the larger drops is a serious drawback to their measurement. Also the stains from adjacent drops often merge to form one large stain. The flour method likewise has several drawbacks. The pellets formed by the larger drops are somewhat irregular in shape and sometimes have to be weighed individually. Again, this method cannot be used under windy conditions as the flour tends to blow out of the containers. In 1950, Blanchard (1)** originated a drop-measuring technique utilizing fine wire mesh coated with acetylene soot. This gave very good results, although some splattering occurred with drops 4.0 mm. or larger in diameter. In looking for a substitute for the wire mesh, dark nylon hosiery mesh was tried. This eliminated all splattering and gave excellent results. The sampling element as now used (Fig. 142) consists of a section of mesh cemented to a hoop 7 1/2 inches in diameter. The mesh is treated with a trace solution of lanolin in naphtha, dried and dusted with confectioner's sugar. Falling drops passing through the mesh clean out the sugar and produce a pattern of good contrast. The screen is then photographed exact size on high contrast paper (Fig. 143). Drop size measurements are made to the nearest tenth of a millimeter from the paper negative, using a wide field low power microscope with a magnification of 7.5 diameters and a scale etched on glass which incorporates the calibration curve of the nylon screen (Fig. 144).

The procedure in sampling rain is to expose a prepared element perpendicular to the rain for a sufficient length of time to collect a sample of 300 to 500 drops, usually a matter of 10 to 30 seconds, less in heavy rains. The element is held firmly in the hand at arm's length, and the exposure time is controlled by a cardboard shutter. The screens

FIG. 142. —NYLON SAMPLING ELEMENT SHOWING DETACHABLE PLASTIC DIAPHRAGM.

*Research Meteorologist, Mt. Washington Observatory, 102 Mt. Auburn St., Cambridge, Massachusetts.

**Reference number in BIBLIOGRAPHY at end of this paper.

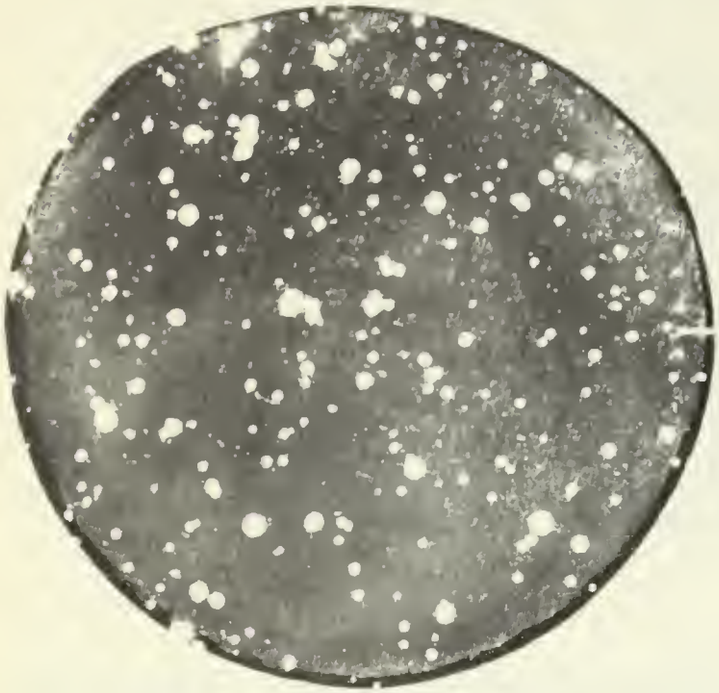

FIG. 143. —PAPER NEGATIVE OF RAINFALL SAMPLE ON NYLON SCREEN.

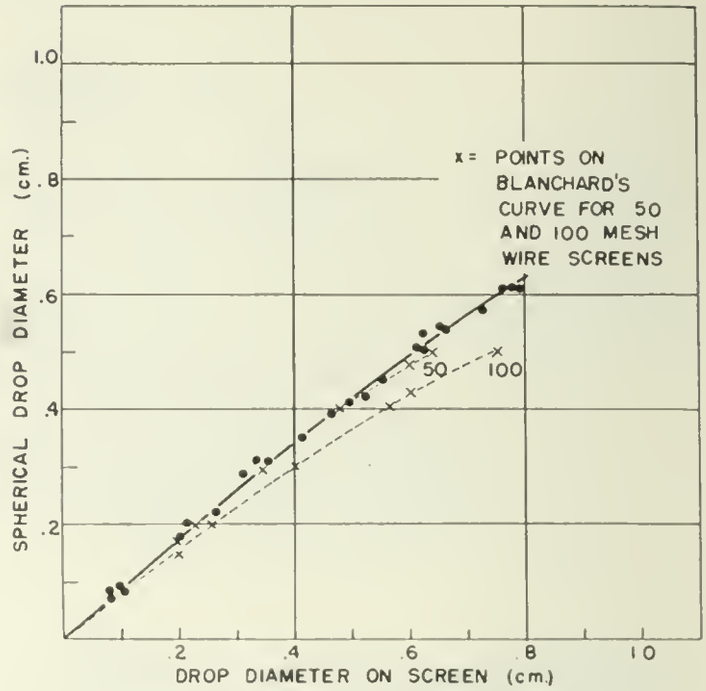

FIG. 144. —NYLON SCREEN CALIBRATION CURVE.

are photographed as soon as possible after exposure to avoid damage to the pattern.

COMPUTATION OF RAINFALL, LIQUID WATER CONTENT AND RADAR REFLECTIVITY

The simultaneous rain intensity is computed from the total volume determined from the drop count. A rainfall check has been made for 50 samples using a plastic diaphragm, shown in Fig. 142, immediately under the screen to retain the catch. By weighing the entire assembly immediately before and after exposure a value of rainfall can be determined from the net weight. In 50 samples the mean difference in the weight of the catch by the two methods was 0.046 g., the average value of the weighing method being 17% lower than that determined by the drop count. Since these 50 samples consisted of low and moderate intensity rain, it is believed that most of the difference in weight can be explained by evaporation losses between exposure and weighing.

Using the distribution of drop sizes as measured to the nearest tenth of a millimeter, the following quantities are computed: R, the rain intensity in millimeters per hour; W, the liquid water content per cubic meter; and Z, the radar reflectivity per cubic meter, on the basis of the following relationships:

$$R = \frac{1}{At} \sum \frac{\pi}{6} \, nd^3$$

$$W = \frac{1}{At} \sum \frac{\pi}{6} \, \frac{nd^3}{v}$$

$$Z = \frac{1}{At} \sum \frac{nd^6}{v}$$

where A is the screen area in $m.^2$,
t is the exposure time in seconds,
d is the drop diameter in mm.,
v is the terminal velocity in $m.\ sec.^{-1}$.

The computation of the liquid water content W and the radar reflectivity Z may also be done graphically by means of a nomogram and an auxiliary chart at a saving in time but with a loss of accuracy of 3 to 5%. The main advantage of the graphical method is that it provides a single diagram for each sample, showing the contribution of the different drop sizes to W and Z. (Fig. 145)

For the sake of accuracy the values of Z used in this study have been computed, but a diagram has also been plotted for each sample as a check on the computations and in order to have a graphical record of the distribution of W and Z for use in further studies.

RESULTS

Out of a total of nearly 90 rain samples collected at Cambridge, Massachusetts, between March and September of the past year, 63 have been computed and are included in the results presented here.

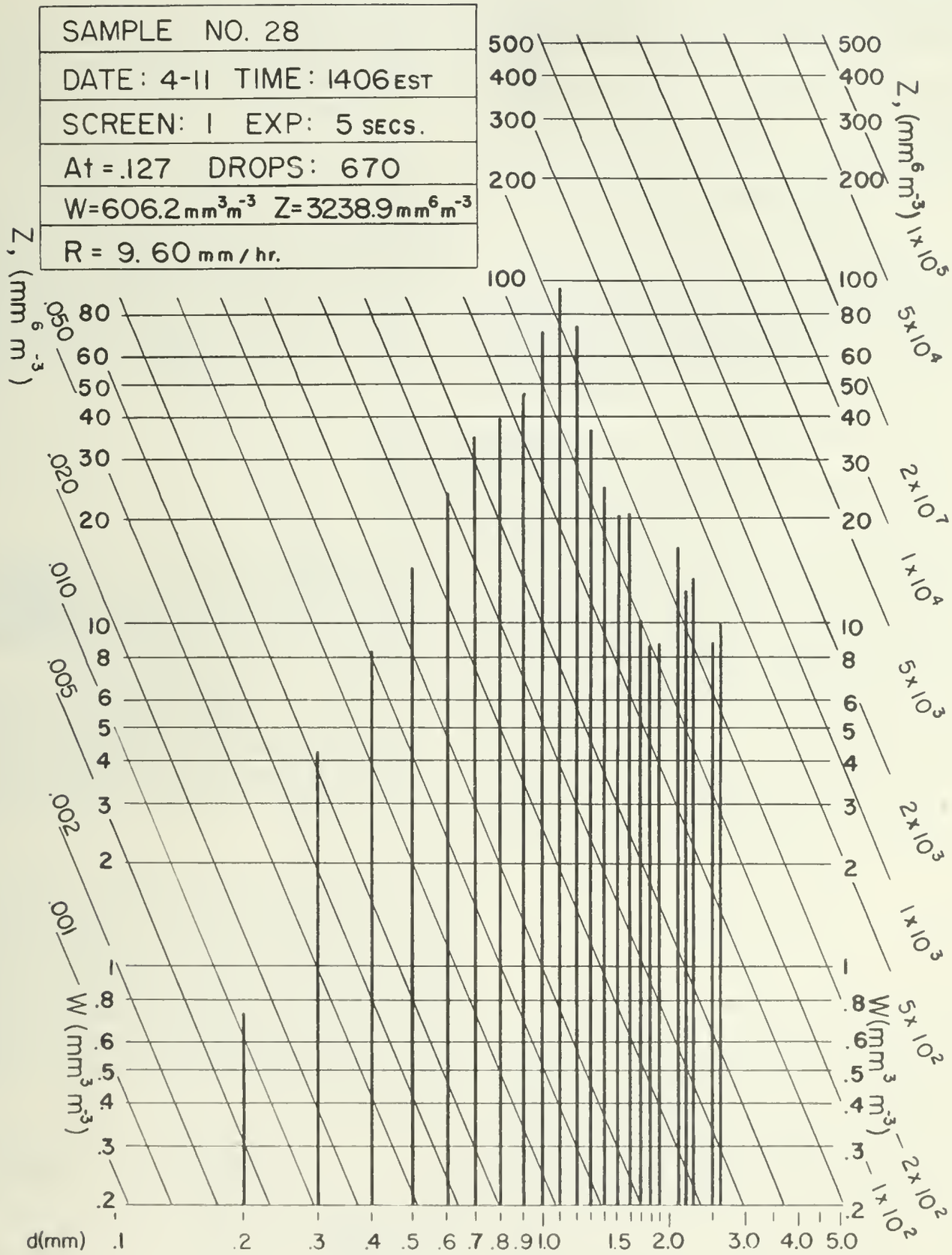

FIG. 145.—NOMOGRAM FOR RAINFALL SAMPLE SHOWING DISTRIBUTION OF W AND Z AT DIFFERENT RAIN DROP SIZES.

These samples include all types of rain: continuous frontal rains associated with coastal storms (some of these were of uniform intensity, others quite variable), spring and summer showers with and without fronts, and a number of thunderstorm rains.

From the data computed from these 63 samples, the regression line of log R and log Z was determined as:

$$\log R = 0.6449; \log Z = 1.5667; \text{ or}$$

$$Z = 269\ R^{1.55}.$$

Fig. 146 shows a plot of R versus Z on a log-log scale, and the regression line computed from the above equation. For comparison, equations computed by Wexler (2) for other data are given below:

from Anderson's data for orographic rain in Hawaii: $Z = 208\ R^{1.53}$;

Laws and Parsons' data from Washington, D. C., gives: $Z = 214\ R^{1.58}$;

while Marshall and Palmer, working near Montreal, Canada, found: $Z = 220\ R^{1.60}$;

and Hood (4) near Toronto, Canada, found: $Z = 295\ R^{1.61}$.

The exponents of R are in very close agreement, which fixes the slope of the regression line within fairly close limits. The variations in the coefficient of R are larger, but the difference in the value of computed R by substituting one coefficient for another is small. For instance, if the coefficients 269 and 214 are used to compute two values of R, these will be within about 10% of each other. The most discouraging factor in this study is the large value of the standard error. In this case, using all 63 samples, the standard error

$$S_R = 47\%,$$

$$\text{where } S_R^2 = \frac{1}{N} \sum \frac{(R - R_z)^2}{R_z},$$

where R_z is computed from $Z = 295\ R^{1.61}$.

The use of log Z and log R in obtaining the regression equation is justified purely for convenience since it allows the use of linear regression methods. But, unfortunately, if there is appreciable scatter about the line, the standard error determined from linear deviations about the regression line will be too large since the line is not the best fit from the point of view of the least squares of the linear deviations.

By trial and error a value of the coefficient of R which determines the locus of the line giving approximately the lowest value of the standard error has been found to be 180, with a corresponding standard error of 38%.

In an effort to determine the horizontal homogeneity of various types of rain, a series of pairs of simultaneous samples is being undertaken. So far only one pair of samples has been analyzed. This was taken during a light shower on July 12. This sample has a broad, flat distribution of drop sizes—the type of distribution where large differences would be most likely to exist between samples. Here are the data for the two samples, both with an exposure time of 12 seconds:

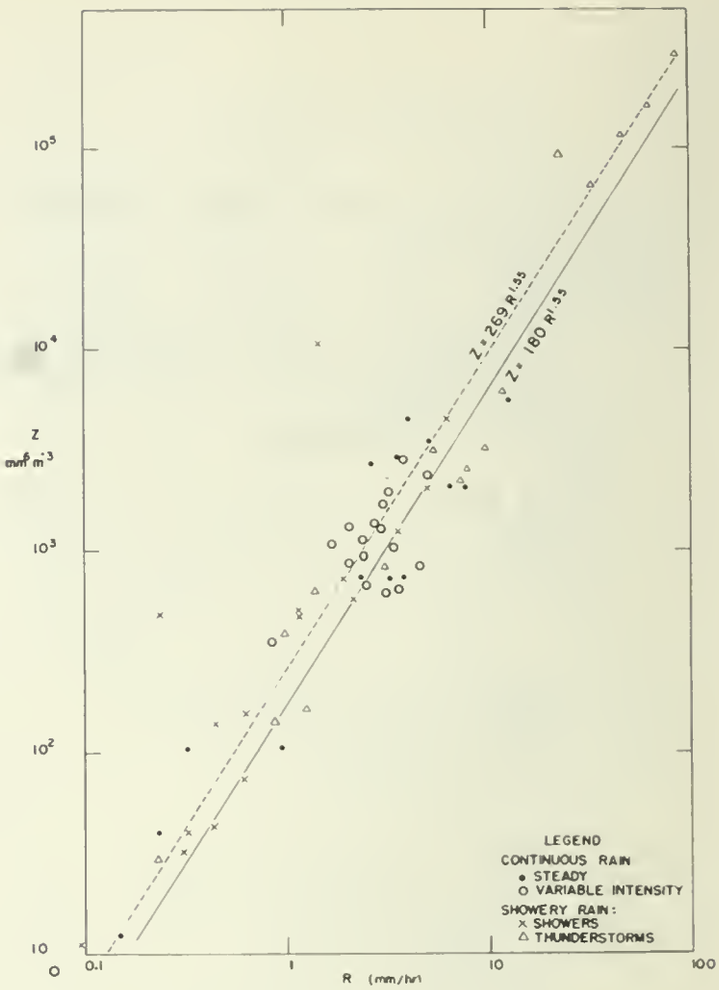

FIG. 146.—Z - R RELATIONSHIP FOR 63 RAIN SAMPLES TAKEN AT CAMBRIDGE, MASS.

The dashed line is the least squares regression of log R on log Z; the solid line is the best fit for least standard error.

	Sample No. 1	Sample No. 2
No. of drops	318	373
Mean drop diameter	1.14 mm.	1.12 mm.
Standard deviation	4.45 mm.	4.79 mm.
Rainfall	5.67 mm./hr.	6.15 mm./hr.
Z	3880 $mm.^6 m.^{-3}$	4510 $mm.^6 m.^{-3}$

The two drop-size distributions were subjected to a statistical "t" test and the difference was found to be very insignificant, indicating the rain in this instance to be homogeneous over a distance of 100 feet. Further sampling of this type will be made to determine the horizontal variability of rain in different types of precipitation over distances ranging up to 400 or 500 feet.

CONCLUSION

The nylon screen technique is a very satisfactory and accurate method of making drop-size measurements in rain, eliminating most of the disadvantages of the previous methods. The results of 63 samples collected at Cambridge between March and September, 1951, yielded a value of

$$Z = 269\ R^{1.55},$$

which is in close agreement with the results of previous investigators. The standard error of R of 47% can be reduced by obtaining a better-fitting line,

$$Z = 180\ R^{1.55},$$

the standard error for this equation being 38%.

ACKNOWLEDGMENT

The research reported in this paper has been made possible through the support and sponsorship extended by the Geophysics Research Division of the Air Force Cambridge Research Center, under Contract No. AF 19(122)-399.

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Ref.

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DISCUSSION

R. J. BOUCHER. —(In reply to Mr. Jones' query.) On the Z-R diagram, Fig. 146, the rainfall is shown classified into four categories: continuous steady rain, continuous rain of variable intensity, showers and thunderstorms. This is a first and rough attempt to differentiate between types of rain. There appears to be no simple correlation between the type of rain as shown here and the departure of the points from the regression line. However, further consideration will be given to the variability of Z and R in relation to observable differences in atmospheric processes.

R. WEXLER. —It is interesting to note here the variation in the exponent of R in the regression equation. For 53 cases with maximum precipitation intensity of 12.4 mm./hr. the exponent of R was 1.38. With more samples, which included rain intensities as high as 87.5 mm./hr., the exponent for 63 cases was raised to 1.55.

NEW METHOD TO MEASURE RAINDROP SIZE

BY L. G. SMITH*

In Cambridge (England) we have been using a new method to measure the size of raindrops. It is to measure the change in capacity of a parallel-plate condenser when a drop falls between the plates. The fractional change in the capacity, C, caused by a spherical drop of volume V_d is

$$dC/C = 3V_d/V_c,$$

for $V_d \ll V_c$, where V_c is the volume of the condenser and the dielectric constant of the material of the drop is large compared with unity. The condenser used had plates 10 cm. by 12 cm. and a separation of 7 cm. The change of capacity is small, being about one part in 4×10^3 for a drop of 5 mm. diameter, necessitating a sensitive method of detection. The method finally chosen, and which has been found to work very satisfactorily, is to make the condenser part of the tuned circuit of an oscillator working at 120 Mcps., the effect of the drop being to produce a change of frequency. The signal is reduced in frequency first to 10 Mcps. and then to 0.5 Mcps., the frequency deviation being unaltered, and detected in a phase discriminator. In this way a voltage pulse is obtained which is proportional in magnitude to the mass of the drop. If the apparatus is set to measure drops up to five or six millimeters in diameter, the smallest size that can be detected is rather under one millimeter, representing a range of mass of 100 to 1. A separate method had to be used to measure the smaller drops.

The main difficulty encountered in the construction of the apparatus was its sensitivity to vibration but this was largely overcome by making the condenser and associated oscillator very rigid. A discussion of the observations taken with this apparatus would be out of place here as they were part of an investigation of the relation between the electric charge and size of raindrops; the drop-size distribution was a secondary feature.

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A PANEL DISCUSSION

NEW DEVELOPMENTS IN USING RADAR FOR HURRICANE TRACKING

ROY C. JORGENSEN,* Moderator

I am partly responsible for a program of this kind and would like to describe my reasons for suggesting it. We at Dow would like to see a yearly meeting of everyone who actually has a radar set in operation for the purpose of hurricane detection. Further, I would like an opportunity to visit all the other sets, and would like other operators to have the opportunity of visiting our set to exchange ideas on radar photography, scope interpretations, and experimentation; in other words, a mutual cooperative program against the common enemy, the hurricane. We at Dow are interested in anything that will aid in the detection of the hurricane. As far as that goes, if microseisms were usable in our area, we would certainly be interested in working in that field also. I hope to stop in St. Louis on my way back to gather information on this subject. We would like to acquaint other people with the work we are doing and to learn what they are doing in order to aid each other in this common work.

*Electrochemical Engineer, Dow Chemical Co., Freeport, Texas.

UNIVERSITY OF FLORIDA RADAR INSTALLATION

BY MARINOS H. LATOUR*

The University of Florida weather radar station has a ten-centimeter wave length radar, the SCR 615B, as its principal equipment. We have just recently moved the weather radar station from its original site at a former air base (about 8 miles northeast of Gainesville) to the campus of the University. At the air base the radar antenna was mounted on the original 25-foot tower. Unfortunately, the rapid growth of the slash pines on the air base resulted in the radar beam becoming obstructed at zero degree elevation over most of the northern semicircle. To overcome this problem, and to make the weather radar station more convenient to our other activities, the equipment was moved to the campus this past summer. Utilizing a radio tower relinquished by the University's radio station, we were able to mount the antenna 120 feet above the surface. This clears all buildings, and we have an unobstructed horizon in all directions. We are also planning to reinstall our 3-centimeter radar previously installed at the air base. Another 3-centimeter radar will be installed in a mobile unit so that we can move to a hurricane. Since the average number of storms per year for all of Florida is only a little greater than one, there may be several years during which no storm will pass within range of the fixed radar station.

We were fortunate to have sufficient funds allocated for the moving of the weather radar station and also new buildings for housing the equipment.

The first radar observation of a hurricane was in 1949. We have a very good record of that storm, starting several hours before the storm proper became visible on the scope (showing frequent squall lines) and carrying through until the storm was fairly well dissipated. Unfortunately, during the latter part of the storm when it was actually nearest the station, it had moved into the sector where it was necessary to tilt the elevation of the beam up several degrees to clear the trees near the radar site. Although the back side of the storm was lost by tilting up, we were able to compute the approximate height of the precipitation in the storm. The top of the precipitation echo was found to be about 13,000 feet. This was confirmed, later, by an air reconnaissance report.

In 1950 we had three storms, which ran our average up. The first storm was the Cedar Key hurricane which developed near the Isle of Pines south of Cuba. This storm passed over Cuba and moved very rapidly parallel to the west coast of Florida, about 30 to 60 miles offshore. It seemed to show a tendency to move to the northwest but suddenly it slowed down, made a complete loop, and turned back towards the coast. The storm moved slowly toward the coast, stopping just a few miles south of Cedar Key. It then moved over Cedar Key and back again, making another small loop. It then continued a slow movement to the south.

The storm was in the vicinity of Cedar Key for about 12 hours, and the town experienced hurricane winds for about 12 hours. When the storm moved over Cedar Key and then backed off, that town experienced the same side of the hurricane twice.

The second storm of 1950, which we call the Miami hurricane, also developed south of Cuba. It broke up somewhat moving over Cuba, but then reformed as a very small storm. I have seen photographs of the radar scope at the Naval Station at Key West. These show the storm as a single, tight ring type precipitation echo. The reports from two automatic reporting stations in the Keys showed only light winds when the storm passed between them, although they did indicate the circulation. The winds were so light that the forecasters began to doubt that they did have a storm between the two stations, especially since the distance between the stations was in the order of 50 miles. When the storm moved over Miami at midnight the center was about five miles wide. It had about an equal distance of high winds on either side of the center; so that the storm was about 15 to 20 miles wide, with the hurricane winds falling off rapidly on each side.

This was a really intense storm in Miami. Boxcars were blown off their tracks, and it also turned over some automobiles. There were about 13 radio towers lost in the Miami area. Those towers to the west of Miami were out of the principal zone and escaped without damage.

When this storm moved into our radar range, it had lost most of its characteristic form. As far as we could see, it was just a series of squalls, well in advance of the low pressure center. We plotted up some of the surface data from the stations along the storm's path. This indicated that the highest winds moved forward so that they occurred about 2 hours or 70 to 80 miles ahead of the low pressure center.

The other storm, which developed only to a small extent, was a tropical cyclone in the Gulf of Mexico. This storm moved in toward Tampa, prac-

*Assistant Research Professor, University of Florida, Gainesville, Florida.

tically due east toward the coast. It lost force rapidly due to the influx of cold, dry air while the storm was still in the Gulf. Before reaching the coast it curved to the north, crossing the Florida coast near Cedar Key. There was very little damage in that area, and the wind velocities were not very high.

There was considerable electrical activity in the vicinity of our station during this storm. At the time the previous hurricane passed over Miami, considerable electrical activity was noted near the center of the storm. This is unusual as our experience and our sferics records indicate small or no electrical activity with hurricanes.

(Note: The following comments were made during the showing of the film.)

This shows our old site with a 3-centimeter radar on top of the SCR 615B radar. We use a modified 0-15 camera for photographing the PPI scope. Since we photograph the radar scope using only one mirror, and the data card and clock using two mirrors, the data card and clock will appear as a mirror image on the film.

These show the Cedar Key storm. The center—if you can see it—was about 120 miles from the station when we first detected it. It seemed to be characterized by a single spiral near the "eye." Unfortunately, we have had to delete sections of this film because we had camera trouble and some operator errors. There was much more detail visible than we have recorded on this film.

These are the radar movies of the Miami hurricane. We started out getting what appeared to be a typical outer band and expected to see successive bands moving in behind this as the center moved towards the station. Actually, we did get this pair of bands—and some finer precipitation in this region.

These are the films of the 1949 hurricane. Early in the afternoon of the day preceding the storm, we saw this squall line and this smaller squall line passing over the station.

Here, about 4:00 a.m. we can see the "eye" of the storm on the scope at about 120 miles. Notice that this large area appears to move around the "eye"—the "eye" appears to be near the west side of that open area. We found that there was a tendency to call any area with no precipitation the "eye" of the storms. The shape of the spirals and the previous appearance have to be considered in selecting the "eye" of the storm.

During this period we turned the dish up in elevation so that we would clear the nearby trees. The center of rotation can still be picked out, you will notice.

This map shows a suggested radar network for Florida. Stations now exist at Tampa and Miami—as well as Gainesville, of course.

(Note: This is the end of the film showing.)

We feel there can be no question as to the usefulness of radar for storm location and detection. The problems now are how to get greater utility from the equipment. It should be possible to get more than just information as to location and movement. There are some problems which have come up. There seem to be discrepancies between points selected as the center of the storm from the radar bands and the actual center of lowest pressure. There is also some question about the movement of these storms. They do not seem to move along a smooth curve but seem to oscillate about the smooth path.

These are problems which we feel require further study.

DISCUSSION

BY

F. C. WHITE, D. ATLAS, J. S. MARSHALL, H. R. BYERS, J. R. ANDERSON,
M. H. LIGDA, J. C. FREEMAN, R. C. JORGENSEN
L. J. BATTAN

R. C. JORGENSEN. —I would like to introduce the panel: We have Mr. Anderson, who was our meteorologist at Dow Chemical Company until the time he went to work for his Uncle Sam; Mr. Latour, from the University of Florida; and Mr. Dunn, of the U. S. Weather Bureau. I would also like to consider everyone present as panel members for this discussion. We will go down the list of topics, one at a time, and suggest that the panel and audience join the discussions. There is one restriction—we have 45 minutes left before the next speaker and about eight subjects to touch on. It is likely that we will have to chop some subjects short, for it is better to at least mention each topic than to leave one completely out. Mr. Latour suggested a division in these topics, the first being radar equipment, and the second being utilization of radar for storm location.

PHOTOGRAPHIC EQUIPMENT

Fig. 147 is a photograph of the ceiling of our operating room, showing two of our cameras. The camera on the right is the one used by most companies to photograph employees for identification badges. It holds 750 exposures of 35 mm. film and is used as a still camera. Exposure is made by opening the shutter for one revolution of the antenna. We usually take a still photograph about every 15 minutes. The other camera is an aircraft gun camera, which is modified to expose one 16 mm. frame during each revolution of the antenna. This camera produces a movie film in which all movement is speeded up about 500 times normal speed.

UNIDENTIFIED.* —What is the film exposure time?

R. C. JORGENSEN. —One complete revolution of the scope, which gives an exposure of about 15 seconds.

UNIDENTIFIED. —Do you have a clock or date card at the scope?

R. C. JORGENSEN. —We have a clock and a date card, which can be seen in the upper right-hand corner of each scope photograph. The camera shown on the right in Fig. 147 photographs the auxiliary scope, which is at the top of the operating panel. This scope will be seen in the upper left-hand corner of Fig. 148. The bottom of the scope is about 6 feet high, and the camera is above 6 feet so that a person can walk under the camera's line of view without interfering with the photographic process. Visitors can view the scope from close range without shading the scope from the camera. Visitors to the radar during emergency operation have been a major problem. They upset operating schedules when we had only one PPI scope, but now we have made arrangements for visitors to view the elevated auxiliary scope, which is also used for photography.

The third camera (Fig. 149) is homemade, takes an 8" x 10" negative, and uses a projector lens swiped off of a photographic projector. We have this camera mounted so it looks at the new PPI scope, which can be seen at the right center of Fig. 148, and which is part of the SCR 527 console. With this camera we gather data for our experimental program. By this means, we get 8" x 10" negatives, which are large enough to be placed one above the other and viewed over a ground glass in order to compare successive photographs. We have also arranged facilities for developing these films in 10 minutes.

F. C. WHITE. —For work on Bureau of Aeronautics, Navy Contract, American Airlines was provided a Bell and Howell "Eymo" camera. It had provisions for the use of a 400-foot spool of film. The camera had a "stop frame" attachment, that is, it would take one frame at a time. Roughly speaking, this provides approximately five thousand pictures of the radar scope for each film lead. The camera had a convenient carrying case; in fact, I carried it to Newfoundland and England and obtained excellent photographs of radar scopes.

R. C. JORGENSEN. —Was it 16 mm.?

F. C. WHITE. —No, 35 mm. The larger size film provides a negative that can be blown up to 12" square, if need be, with good detail provided.

*During the discussion there were comments by persons who were unidentified in the recording.

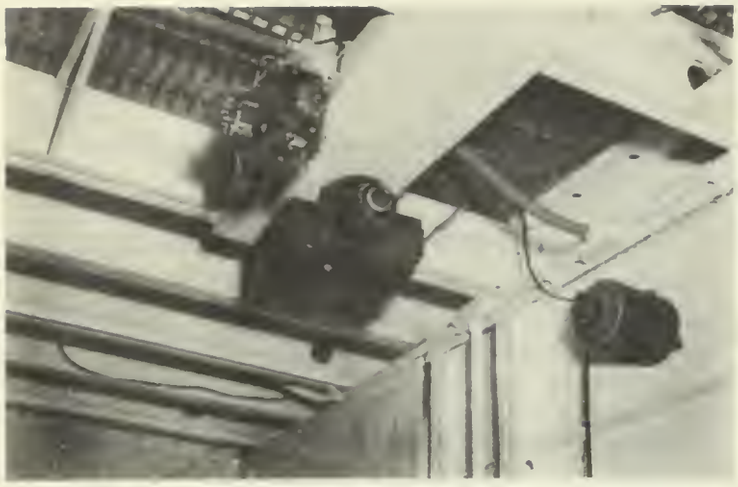

FIG. 147. —CAMERAS IN CEILING OF RADAR OPERATING ROOM.

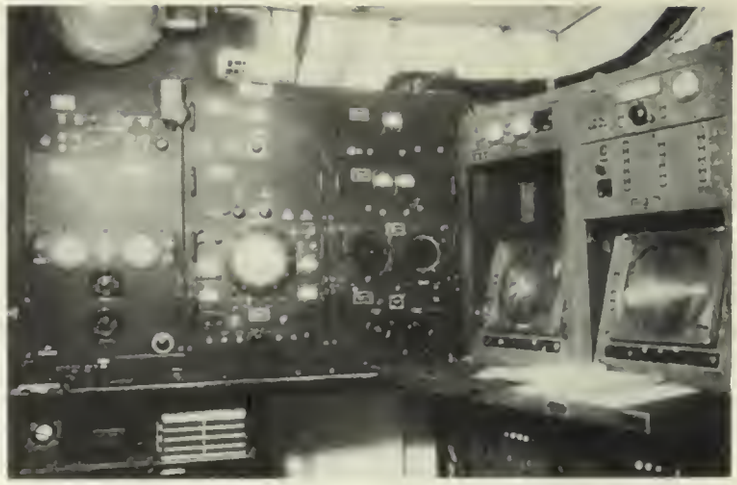

FIG. 148. —RADAR CONSOLE.

PLOTTING BOARDS AND REPORTING HURRICANE LOCATIONS

R. C. JORGENSEN. —Fig. 150 shows our plotting board. It is made up of aircraft navigation maps placed together, and we have the latitude and the longitude shown on the side and bottom of the map. This map is the best we could find which would allow you to measure distances directly on the surface of the map with the least amount of distortion. We have a range arm out to 300 miles. The map extension protruding at the center right of the plotting board is the Humble radar site, which has discontinued operation as of this year. This radar set was located at Grand Island, Louisiana, and we regret that the station was closed. We would like to see somebody continue operating the equipment for hurricane tracking. The face of the plotting board has been covered with thin plastic, and we can mark on it with wax pencils, as shown by the rain area plotted in Fig. 150. Markings on this plastic can be easily removed with a soft rag. It makes a very nice plotting setup. It keeps your map clean, and you can do all the marking and erasing desired without permanently defacing the map. I noticed today that the Illinois State Water Survey is using the same system on the display board about radar-weather. Anyone have suggestions on this topic?

UNIDENTIFIED. —What do you usually plot? The center, or echoes, or what?

R. C. JORGENSEN. —When plotting hurricane tracks (the only weather phenomena in which we are really interested), we carry two plotting references continuously. These are as follows:

1. We plot the estimated center of the eye, which is quite difficult to determine accurately.

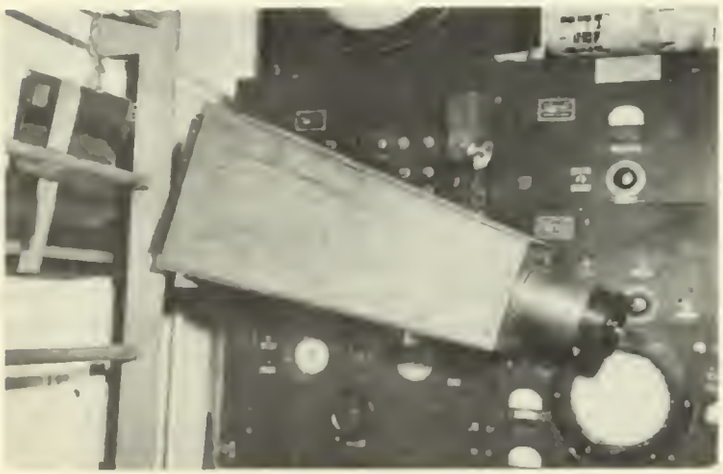

FIG. 149. —CAMERA USED TO OBTAIN 8" x 10" NEGATIVES.

FIG. 150. —PLOTTING RAIN AREAS.

2. In addition to the above, we also plot a point on the inside edge of the smallest arc of the hurricane pattern. We feel that this point can be followed much more accurately for the purpose of determining hurricane speed.

We have a new system using negative overlays which we intend to put into operation during the next hurricane. This negative overlay system will be discussed later. The two plotting points mentioned above are carried in different colored pencils to more or less check one against the other. We find that the average speed derived from a series of plots gives a fairly accurate speed of movement for the hurricane. Most of the difficulty encountered in determining hurricane speed of movement and eye location originates from the continually changing shape of the hurricane arcs. Most of them just do not stay put. They change—there will be a large center arc and then it will form a little hook at the end. You do not know whether to take the circle from the little hook or use the entire arc for locating the eye.

Incidentally, there is one suggestion we have that we would like to pass around to all radar hurricane operators. Being a landlubber, I can not help but think in terms of statute miles, which is the system used on our set. Some others are calibrated in nautical miles. For fear of confusion of the two systems, we have decided to convert all data to latitude and longitude before transmission to the Weather Bureau. We can plot data on our board using range and azimuth, and then convert it to latitude and longitude very easily because we have both available on the plotting board. This conversion is conveniently accomplished on a map of this type. In the past, it has been necessary for the Weather Bureau to have a special plotting board, such as that shown in Fig. 150, in order to use the information. If the information is given in latitude and longitude, as we suggest, no special equipment will be required to plot radar hurricane information.

UNIDENTIFIED. —Do you think that a system of plotting using latitude and longitude might encourage some Government agencies to cooperate in the radar hurricane network?

R. C. JORGENSEN. —That is correct. It may lead to getting a little more cooperation from stations not allowed to disclose their locations.

D. ATLAS. —In order to observe storm movement you might be interested in using the recording storage tube. You could then record and retain the PPI patterns at 5- or 10-minute intervals for comparison purposes.

R. C. JORGENSEN. —Is that commercially available?

D. ATLAS. —I am not quite sure if it is on the market yet, although you can undoubtedly get them from Raytheon on special order.

IMPROVING RECEIVER SENSITIVITY AND INCREASING THE USABLE RANGE OF EQUIPMENT

R. C. JORGENSEN. —The set of curves in Fig. 151 shows improvements made in receiver gain. The dotted curves can all be compared with each other, and the solid curves can be compared with each other, but the dotted curves cannot be compared directly with the solid curves due to a change in oscillograph sensitivity. The original set had a two-stage pre-amplifier, and we replaced it with a four-stage pre-amplifier. The dotted line is the response curve of this four-stage pre-amplifier, which we purchased from Terpening and Company. The original pre-amplifier had a very long flat curve similar to the dotted curves, but it is not plotted on the chart. The dotted lines show the response of individual components before tuning. Fig. 151 shows that the main receiver and the remote video chassis contribute very little to the over-all response. While this wide band receiver design was ideal for use in the original automatic tracking and gunlaying equipment, it was found to be undesirable for hurricane tracking radar. It was found that the wide band frequency response of this receiver encouraged noise reception over a large area of the spectrum, while affording only a minimum amount of gain for desirable targets. To correct this situation, the remote video chassis and the main receiver chassis were re-peaked and re-aligned to produce the tall, solid-line curve shown in Fig. 151, which gave approximately five or six times the over-all gain found in the original unit, and at the same time eliminated at least seventy percent of the side band noise.

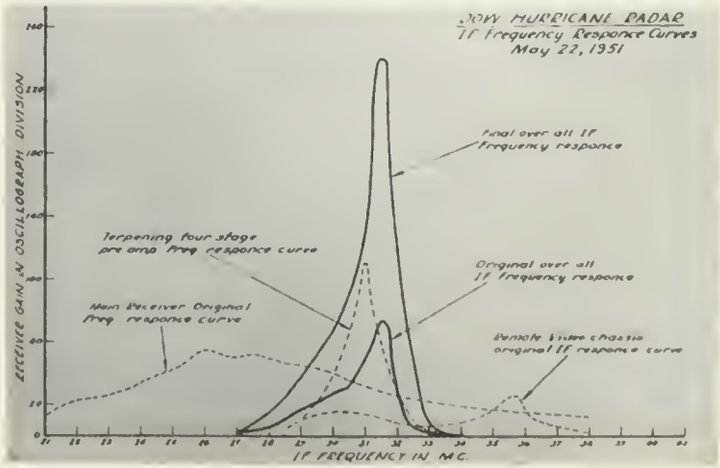

FIG. 151. —IF FREQUENCY RESPONSE CURVES.

We feel that we have made a large step forward in improving the set by retuning these I.F. channels. It took more than screwdriver adjustment. We had to actually melt the wax on the coils and bunch the coils together, or spread them out, or add turns, etc. to peak them and move their frequencies far enough to achieve our desired purpose. It was only after these modifications that we had targets at fantastic ranges over 300 miles, and I give tuning and I.F. peaking full credit for making this possible. I understand that some of the sets you other gentlemen are using also have been changed in a similar manner for long range work.

The original pulse length was 0.8 microseconds but we removed the delay line and now have a pulse length of about 1.6 microseconds.

J. S. MARSHALL.—With that pulse length, was there any effort made to carry it farther?

R. C. JORGENSEN.—No, there was not. We are quite pleased with the improvement we have made to the present time. Perhaps we will have enough nerve to try something else if we find a need for more gain.

J. S. MARSHALL.—It is interesting to see how far you can carry it before you cease to make further improvement.

R. C. JORGENSEN.—The band pass now extends from about 30 1/2 mc. to 31 3/4 mc., which is a fairly narrow frequency response. I would hesitate to attempt to carry this work further.

UNIDENTIFIED.—Can this band pass be made sharper than you now have it?

R. C. JORGENSEN.—Yes, it can be done. We were afraid to go too far with it. The band pass of the original unit covered from 21 mc. to 37 mc. and we have reduced it to about 1 1/2 mc. That is a pretty big step, and we feel we have gone about as far as we dare.

UNIDENTIFIED.—Do you have a device for checking over-all operation?

R. C. JORGENSEN.—We have an echo box, but the only method we have of using the equipment is by putting a dipole in front of the antenna. The antenna is 30 feet above the roof and is quite inconvenient to reach from the operating room. We have our I.F. units tuned up at the present time. This was carried on stage by stage and then unit by unit in the tune-up, and we had quite an elaborate array of equipment. We did not have the necessary alignment instruments, and we found it cheaper to hire someone to come in, bring the equipment, and assist in the alignment. Straughn Radio and Electronics Company, of Beaumont, Texas, furnished the needed test equipment and technical assistance for this job.

F. C. WHITE.—Fixed targets are very useful for making "loop gain" tests. They provide you with information concerning both transmitter output and receiver sensitivity. You stop your antenna, point it at some "fixed target"—a good strong echo at less than 15 miles range—and reduce receiver gain until the target just disappears on the scope. You record what gain setting resulted and by making day-to-day checks can quickly determine if system performance is constant or on the decline.

R. C. JORGENSEN.—That is the method we use. We have a perfect target 20 miles from the station. It is a horseshoe made up of trees, oil

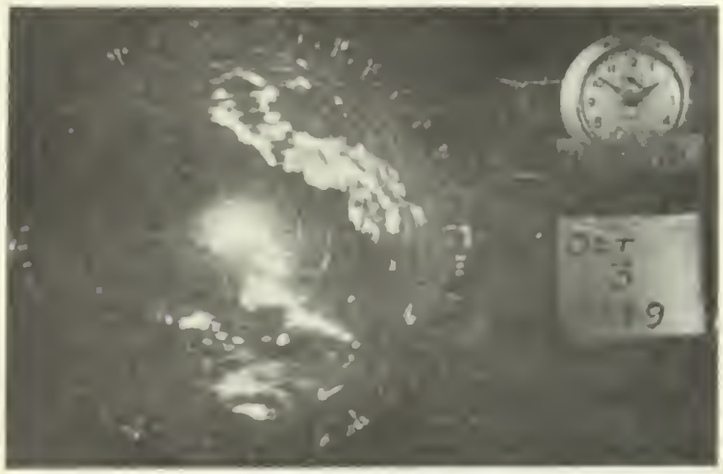

FIG. 152.—PRINT FROM 8" x 10" PLATE BACK CAMERA.

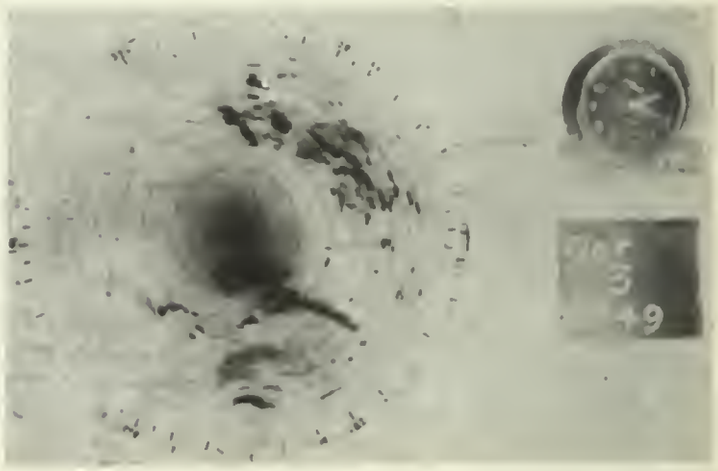

FIG. 153.—PRINT FROM 8" x 10" PLATE BACK CAMERA.

wells and a town. When the set is poorly tuned, all we get is the mouth of the horseshoe made up of reflections from the oil wells. When it is tuned up a little better, we can get the two legs of the horseshoe, consisting of echoes from oil wells and tall trees, but we do not get the back side of the horseshoe unless the unit is well tuned. When we get the full horseshoe, the set is really on the button. It takes very careful tuning to get the complete horseshoe on the scope.

UNIDENTIFIED. —What distance is this tuning target from the radar?

R. C. JORGENSEN. —It is only 20 miles in a direct line of sight, and we can see it from the radar tower on a clear day.

D. ATLAS. —Although your equipment appears to be operating beautifully, I would be cautious in using ground targets as a standard of reference. Under trapping conditions your ground targets may show up fine when the set is operating well below par.

THE UTILIZATION OF RADAR FOR STORM LOCATION

R. C. JORGENSEN. —We have devised methods of determining rate and direction of storm movement from the radar scope presentation. What we do is take a photograph of the scope and make a positive on transparent film, such as shown in Fig. 152, using an 8" x 10" plate back camera. We make a positive, and then later on we make a negative, such as Fig. 153, which is a half-hour different in time than the previous one. When you lay these two, one on top of the other, the parts that are dark on one are light on the other and vice versa. The result is that if there has been no movement and everything remained identical during that half-hour period, everything would have blanked out and looked like two black cats fighting in a dark alley. But if there has been some movement, it will stand out like a sore thumb.

Fig. 154 demonstrates a composite presentation of a positive and negative transparency laid on top of each other, and you will notice that the range markers and everything else are blanked out, and the width of the clear space is the exact movement which has taken place in that period. You can see the amount of movement of the eye and of every small squall. It will outline every movement accurately. By taking the movement distance with dividers and applying it to the range markers, we get the exact distance of movement. It looks like this system has quite a bit of promise.

H. R. BYERS. —What is the splotch?

R. C. JORGENSEN. —That is a squall ahead of the hurricane. This was a large rain area which dissipated. Since this was a change, it showed up on this overlay method. Since rain areas which dissipate constitute a change from the previous photograph, it will show up in the same manner as a rain squall which merely moved from place to place. Dissipated areas can be identified from actual movement by visual inspection of the two films. But in the case of the eye, it has remained almost the same shape and movement is shown. The same applies to the outer ring. The movement is obvious. I think you will find this method very useful. In the same way, we hope to take a very small indentation or knot on one of the bands of rain, select one, say 30 miles from the center, 50 miles from the center, or 100 miles from the center while the storm is still 200 or 250 miles from the radar, and get the exact movement of a small knob on a rainstorm and the velocity of rotation of the rainstorm itself.

Mr. Anderson has a theory that would make use of this information in estimating the maximum wind at that distance from the core of the hurricane. The ultimate would be the possibility of determining the wind at different distances from the center. Knowing the speed of movement at various distances from the center, and also knowing the rate of movement of the hurricane, it would be possible to predict the future wind velocity on the radar site several hours in advance. This information would allow more intelligent shutting down of the plant, knowing what wind velocity the buildings can stand. The plant, of course, must be completely shut down before the outer ring of the eye gets there, and that is some distance from the center of the eye. Our main concern is to know when dangerous winds will reach the plant. This is more important than where

FIG. 154. —POSITIVE AND NEGATIVE TRANSPARENCIES LAID ON TOP OF EACH OTHER, SO WHITE AREA IS MOVEMENT OF STORM.

the center of the eye is located. What is the wind going to be six hours from now? five hours from now? and when shall we shut down in order to have everything cleared by the time the wind gets too strong?

UNIDENTIFIED. —You could get errors in movement from dissipation of rain areas, could you not?

R. C. JORGENSEN. —That is right. Areas of dissipating rain can be distinguished from actual movement of rain areas by visually comparing individual transparencies. Movement information gained from these overlays cannot be taken blindly but require a certain amount of inspection and interpretation.

J. R. ANDERSON. —The movement of the inner arc is in our experience consistent with the movement of the storm itself. The distribution of the outermost arcs which are not in the circulation of the storm proper bear no consistent position relationship to the storm center. Determinations of the movements of these outer arcs give widely varying results over short periods of time, although there is at times evidence of a qualitative relationship between their movement and that of the storm. Errors in movement may be induced by variations in the structure of the storm center, which in turn would cause inconsistencies in the position relationship between the inner arc and the geometrical center. Such variations in structure are considered to be due to a dynamic upsetting of the force equilibrium established in the storm core and are themselves significant.

D. ATLAS. —Have you attempted to correlate the path of the radar center to that of the pressure center?

J. R. ANDERSON. —Our data are too limited for that. In 1945 in Florida the hurricane retained its basic structure while passing over a considerable number of reporting stations. We had only two stations to use; however, there were indications that the radar and pressure centers were not consistent and that the radar center stopped for periods while the pressure center maintained a slow but steady movement.

R. C. JORGENSEN. —We have one more suggestion for use of overlays such as we have just presented. Our photographer at Dow got together with the Eastman people in an attempt to work out a system of dyeing negatives different colors which might allow us to examine as many as four transparencies (each being a different color) by stacking them one on top of the other and viewing the entire group at one time. By this method it is hoped that movement can be followed by a combination of colors blended from transparence to transparency in such a manner as to describe the exact movement of the rain area in question. This is a completely experimental approach, and nothing definite is available at the present time on the use of this system.

UNIDENTIFIED. —I should use projectors and have different colored filters on the projectors. Have a blue filter on one and a red on another, and what you will get out is yellow where the storm coincides. Red and green where the thing moves.

R. C. JORGENSEN. —Well, it would be pretty hard to line them all up for they must be perfectly superimposed on each other. If we had transparencies such as described above and could lay them one on top of the other, with the ground glass under them, it would be more convenient. If we cannot get colors on the negative, then projection would certainly be an interim measure which could be tried.

UNIDENTIFIED. —Would you have trouble lining them up?

R. C. JORGENSEN. —Each projector would have to be moved so that each azimuth ring fell exactly on the other azimuth ring. Just try to line two of them up like that. I can tell you it is a headache, and I am afraid you will need micrometer screws on the projectors to keep them together, and the least little vibration will throw them apart. If they are not in exact alignment, it means nothing because you are looking for very minute echo movements in 15 minutes of storm movement.

VERTICAL STRUCTURE OF HURRICANE USING VERTICAL SCAN

We do not have a TPS-10 radar. It is the ideal answer to vertical scan. We had the bright idea of using existing equipment to get a small facsimile of the TPS-10 radar by adding a vertical scan switch which will produce a vertical scan presentation as shown in Fig. 155.

One of the PPI scopes is arranged so that while it is still rotating we can flip a switch and change the Selsyn controls of this scope from the azimuth to the elevation Selsyn system, and as we mechanically raise the antenna in elevation, we get a cross-sectional view of the hurricane. On this particular view, a base line has been flashed to give a zero axis. You can note the arcs in the photograph which were formed by the range markers and which give a scale that can be used in both vertical and horizontal measurements.

Fig. 156 shows the same scope except that I

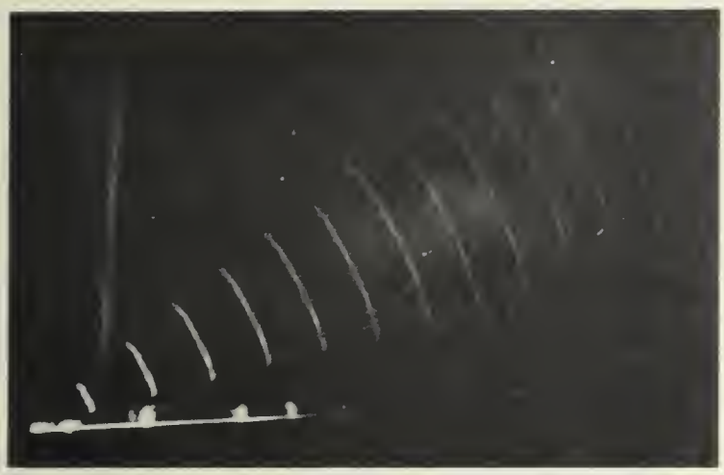

FIG. 155. —RHI PRESENTATION ON PPI SCAN WITH BASE LINE.

FIG. 156. —RHI PRESENTATION ON PPI SCAN WITHOUT BASE LINE.

did not artificially add the base line. I might add that it is "Hail Columbia" to try to get a picture of our vertical sweep because we seem to get so much afterglow on the scope. I got these photographs by cutting the scope off and then opening the camera and photographing the afterglow fluorescence on the tube, but in viewing the scope, the lines are actually much sharper than shown in this figure. I do not have any of our unusual storm cross-sections to show you at this time, but we find some crazy shapes on vertical scans.

The actual presentations of the radar scope are much larger than those shown in Fig. 155 and 156 when a severe rainstorm is being observed. The reason these presentations are small comes from the fact that we have such good weather in Texas I was unable to find a severe rainstorm to photograph. The presentation described above is quite small compared to the enlarged presentation given by the TPS-10 radar. The TPS-10 radar would not serve our purpose due to its short range. Our system is simple and requires only three wires and a switch, using equipment on hand. So we do have a haphazard means of getting vertical scan, and it can be used all the way to 300 miles. Of course, the longer range you have, the closer together the range markers are and the shorter the echo would be. But it is something that is a help, even though it is not the most desirable answer.

We feel that there may be some tie-in between the vertical slant of a hurricane and the direction it is moving. A change in slope may mean that the hurricane is slowing up and getting ready to turn, and it might possibly start leaning in a new direction before it actually turns in that new direction. If this assumption is true, we may be able to predict a new direction of movement before it actually takes place. I realize that this vertical scan is not sufficient in itself, but we believe that with a series of photographs taken on large negatives at different elevations, then laid one on top of the other, you could see the step-over of the movement. For instance, if the hurricane was not leaning straight toward us or away from us, but at a 45-degree angle, this overlay method would show a true picture of slant, and we could take these pictures as fast as we could and ten minutes later we would have the overlay. By this method, we could see the actual direction it was leaning.

We are thinking of taking an overlay every hour and a vertical scan more often. When we suspect a change in slant, we will make a series of photographs and actually determine the slant angle. I must admit that with the small elevation changes we are working with, measurements cannot be considered accurate, but they are better than not having any at all. We do find it interesting, and it is surprising some of the weird shapes we have found. We have seen rain stretching out over an area and no rain on the ground. The flat tops of some of these anvil heads make interesting studies. Does anyone have any comments?

L. J. BATTAN. —How high do they extend?

R. C. JORGENSEN. —Truthfully, I do not know. We have not tried to calibrate the vertical scan height. We can read elevation from antenna angle using a graph. There is some error, of course, due to the width of the beam. I am not too sure of the beam width. I think it is about 2 degrees. It was stated as 3 degrees with the rotating conical beam, and by stopping that rotation we think it would reduce it by about one-third, making it about a two-degree beam, but I cannot vouch for that.

M. H. LIGDA. —You can get a 5 to 1 or 10 to 1 Selsyn and by this means expand the ten degrees

vertical sector of your antenna to 50 or 100 degrees on your scope.

R. C. JORGENSEN. —We have thought of that. The trouble is, we have a 16 to 1 Selsyn driving the deflection yoke through reduction gears to give more power. If we should do as you suggest, all vertical lines would become curved lines. The thing that we are looking for is the slope of the precipitation echo, and if we put artificial slope in it, you would have something to subtract in order to know what you were looking at. You must have something more than a different Selsyn ratio. The range must be changed at the rate of some angular function in order to give true perspective. The TPS-10 radar has such a circuit and does give a true perspective.

UNIDENTIFIED. —That is a virgin field as far as hurricane work in getting some good cross-sections. The TPS-10 is infinitely better than nothing at all. We have to find what there is in the way of shear, etc., height and information such as that. So do not be discouraged especially by the fact that the picture is distorted. You should attempt to get as much out of it as possible.

R. C. JORGENSEN. —We intend to make maximum use of this information and at a later date may be able to purchase special equipment for this purpose.

I had an idea that it might be possible to increase the rate of the movement of the vertical scan, and get rid of the distortion by using a sine or cosine cord on the range controls. We have a control on this set which will change the sweep speed, and if we could get some kind of mechanical coupling between the two so that the sweep speed would extend at the proper rate to make the range markers vertical, we would then have a system similar to the TPS-10.

F. C. WHITE. —Have you considered photographing the flash image instead of the persistent image? The persistent image is the result of secondary emission from the phosphor salts deposit on the end of the tube, while the flash image is the direct result of the electron beam striking the phosphor.

R. C. JORGENSEN. —Up to the present time we have not had occasion to desire photographing the vertical scan scope, and this photograph was made for this meeting. Just on the spur of the moment I tried to photograph it, and I ran into a little trouble by having so much brilliance that the halo effect would cause shading and blurring. The camera we use is an open lens type with no shutter and no iris. We take a photograph with this camera by taking the lens cover off and putting the cover back on. I am an amateur so that would be one answer to the difficulties.

M. H. LIGDA. —Put a filter on it.

R. C. JORGENSEN. —That might help. We certainly will try it.

M. H. LIGDA. —Another thing occurred to me about the radar—that maybe Dr. Byers can tell us or can give us his opinion on how valuable the old 10 cm. Beavertail CPS-4, or whatever it was, is for reliability.

H. R. BYERS. —From what we could tell it was just as good as the TPS-10, except that it had to be rotated manually.

R. C. JORGENSEN. —I would like Mr. Anderson to discuss the possibility of determining wind velocity at certain distances from the eye while the hurricane is still quite distant from the radar unit.

J. R. ANDERSON.* —This is just a little scratching around. To start with, the nature of my work and the purposes for which I did this work did not allow me very much scientific purity. What I did is take all the hurricane pictures and try to pick out individual echoes which I could specifically follow from one frame to the next and in which there were not any complications as to locating the same point on the echo such as would be brought in by development or dying out of the echo. Using that system I determined some echo velocities from still photographs by trigonometry and tried to correlate them with the wind velocities which were experienced on the ground when that part of the hurricane passed over a ground station. I had a correlation time lapse of anywhere from 1 1/2 to 5 1/2 hours in which the velocity and intensity of the hurricane or parts of the hurricane could have changed. However, I got a very steady ratio of three to one between the echo velocity and the average wind speed at the ground (by that I mean the average steady value, nothing was attempted with the gust) until I got in right close to the eye; I should say when I came into the hurricane circulation proper. That brings out another idea of mine in which the hurricane circulation is confined to a small area close to the center outside of which is an easterly wave circulation that is affected by the hurricane. You do get a very easily determinable line of damaging winds while looking at the radar pictures, which is borne out in this echo velocity study. It seems that there is a lot of inertia in the wind on the ground which retards its following the increasing velocity aloft, assuming

*Major, U.S.A.F., Base Weather Officer, Fort Sill, Oklahoma.

that it was reflecting the velocities aloft at 1 to 3 until you got into the hurricane. In the hurricane circulation proper then the ratio went down to 5 to 1 with the echo velocity aloft 5 times the average wind speed on the ground, and then it started back down toward 3 to 1 again as the center was approached and the echo velocity gradient aloft decreased. The surface winds picked up to where they were tending to return, I think, to the original three to one ratio by the time the calm area at the center was reached. I did not get to work on but 20 to 30 points because my rules for acception of an echo, in being sure that I had the same echo and continuity as to its center, were very stringent. I do not recall how many photographs there were, but I did not accept more than one in 90. I do feel that the results accurately portray the relationship in this storm.

J. C. FREEMAN. —Which way does the ratio go?

J. R. ANDERSON. —The maximum average wind velocity which we got at the plant associated with the storm was 61 miles per hour.

J. C. FREEMAN. —I mean the echoes at 20 miles per hour.

J. R. ANDERSON. —I got echo velocities as high as 149 miles an hour. But you realize that when I got into that 149 miles per hour velocity, I was near the center and the possibility of going from one still frame to another with any degree of accuracy, using trigonometry, becomes very limited.

M. H. LIGDA. —What was the time interval between pictures?

J. R. ANDERSON. --Oh, it varies anywhere from 2 minutes 15 seconds to 14 minutes 15 seconds. In other words, to pick out those frames in which I was sure of continuity, I could not have the same time interval in my determinations. Sometimes I had an echo in there I could follow for four minutes; sometimes I had an echo I could follow for 15 minutes. If I was not sure of it after, say, 6 minutes, why I would throw the last two minutes out and go back to 4 minutes. This involves another error. When I first started out in this study I did not intend to get anything but a very empirical relationship for determining what point on the radar picture showed the line of incidence of damaging winds and whether I could detect a change of large magnitude in the intensities of those winds. These changes occur frequently in small storms.

(A question on the time lag.)

J. R. ANDERSON. —You see, from the photographs I determined the orientation and range of the echoes used in velocity determination with respect to the center, and disregarding time I plotted all my determinations with respect to their position from the storm center. Then I plotted the path of Freeport through the storm, correlating echo velocities with the wind velocities at Freeport. That is why I say my time lag is anywhere from 1 1/2 to 5 1/2 hours. My values might have been knocked out by one thing I was trying to determine, that is, variations in the wind velocities in the main body of the hurricane.

UNIDENTIFIED. —Will you point out the so-called "bar" of the hurricane?

J. R. ANDERSON. —There was one "bar" on the hurricane behind which there was almost a solid echo in toward the center. When you reached that "bar," your echo velocity jumped up. Inside that "bar" individual echoes were hard to get because I had the thickest echo there; continuity was very difficult. However, it was clearly at that point where the echo velocities jumped up. When that "bar" hit Freeport, the wind velocity went up (discounting a small time lag as being due to the inertia factor aggravated by friction). This "bar" appeared to maintain a rather constant distance from the center. The outermost "bar" which maintained a constant distance from the center turned out to be the point at which you got hurricane winds.

(A question on duration of individual ecnoes.)

J. R. ANDERSON. —It would no longer exist. In a period of half an hour your echo would have dissipated in the divergent zone on the other side. These echoes as individuals do not last any time

FIG. 157. —RAIN ECHOES SHOWING ON THE "A" SCOPE.

at all. They just run through the arc and dissipate on the west side but new ones keep springing up. The squall line itself is relatively stationary. It has a complete change-over of individual echoes in the space of not more than 45 minutes.

D. ATLAS. —By the way, does the echo velocity agree with the wind velocity? I ask this because we have observed echoes propagating along lines perpendicular to a cold front at velocities greatly different from the wind at any level. We recorded echo velocities of 110 mph. when the wind did not exceed 60 or 70 mph. at any level.

HURRICANE RAIN, INTENSITY DETERMINATION USING AN "A" SCOPE

Fig. 157 shows our "A" scope. This scope makes it easy to compare the different rain intensities. You will find that an "A" scope is very handy for that purpose.

TARGETS DROPPED INTO THE HURRICANE BY RECONNAISSANCE PLANES

R. C. JORGENSEN. —We have already mentioned this idea to Captain Durey. It would be very interesting to drop some window into a hurricane (little pieces of tinfoil cut to 1/2 wave length of the radar). During the war, window was dropped over an area to hide the movement of aircraft from the enemy. We had the bright idea that it might possibly be done by having the last reconnaissance plane in the afternoon drop some window in the hurricane, and it might be possible that window would be carried around and complete the back side of this poorly defined eye to give you a better definition of the location of the eye. It also might be very possible that the upward drafts of the hurricane might carry the window to higher elevations where there is no precipitation, making it possible to detect the hurricane from land radar at a greater distance. There also might be a possibility of dropping a wad of window on the outer edge of the hurricane, following it by radar as it is carried to the center, and making a study of wind currents in the hurricane. No one has been quite as enthused as I have, but they have all agreed that it would not hurt to try it, and the window is a stock item of both the Army and the Navy. We would certainly be glad to cooperate in this program any time there is a hurricane in our vicinity. We would like to see it tried and if the Army or Navy would like to try it, we would certainly be interested in the results. I do think that it may be one way of improving the use of radar for hurricane detection. Now, do not get me wrong. I do not hope to follow this window while it is in the rain area. We hope to fill in the non-rain areas of a poorly defined eye, and increase the range at which a hurricane can be detected, due to the window being carried aloft. The Army and Navy would probably know what length of time that window would stay suspended when used on a bombing run to hide the bombers. I believe it would run better than an hour, and if that is the case, if we get the turbulent air of the hurricane with updrafts in the center, as I understand from Waxler's book, it might be possible it would stay as long as six hours. This would mean that if the last plane in the afternoon dropped it, it would give you better

FIG. 158. —THE PRESENT RADAR RADOME.

FIG. 159. —RADAR TOWER DURING CONSTRUCTION.

observation the first six hours of darkness. You could locate the eye accurately and get some very interesting studies of the storm. According to a sketch in Waxler's book on hurricanes, the wind comes in from the sides of the hurricane and outflows over the top. If that were the case, it might possibly carry this window quite high above the center of the storm.

FIG. 160. —TEMPORARY INSTALLATION USED DURING 1949.

I would like to discuss this a little more but we have already infringed on other people. There are some of the gentlemen here who were absent last night, and I would like them to see what our setup looks like. Fig. 158 shows our present radar dome, which encloses the antenna system of our radar unit. Fig. 159 shows our radar tower during construction, and the eight-foot parabolic antenna reflector can be seen before it was enclosed in the fibreglass and plastic dome. Fig. 160 shows the original temporary radar installation which was in use during the time of the 1949 hurricane in the Freeport area. With this new installation it is expected that the radar unit can be operated right through a hurricane without the necessity of shutdown due to wind-loading on the antenna, such as was experienced in 1949. Fig. 148 shows the main control panel of the radar which is used inside of the main building below the antenna tower.

A PANEL DISCUSSION

SUGGESTED FIELDS OF STUDY

A. C. BEMIS, Moderator

A. C. BEMIS. —This is to be an open discussion so feel free to interrupt at all times. I am flattered to be picked as moderator for this discussion; but in case anybody has the mistaken idea that I really know what I am talking about, Frank White suggests I tell you what happened down in Maine a little over a year ago. I had been on the radio on one of the Lowell Institute adult education programs, and apparently listened to in Brooklin, Maine. The following conversation was overheard by a friend of mine in the country store in Brooklin. One feller said, "Hey, Bill, was that Bemis we heard on the radio last night at 6 o'clock?" Bill said, "Sounded like him." "What the hell was he talkin' about?" "Oh, I dunno, suthin' to do with weather and radar and all that. He was carryin' on there quite a while." The first feller says, "Damn it, those guys down there at M. I. T., now I s'pose they've read 'bout all the books they is to read, and I 'magine they knows plenty, but, if you ask me, they don't realize nothin'." So keep in mind that I may have read some books but "I don't realize nothing."

However to help start the discussion, I have written on the blackboard five fields of study which seem to me particularly important at this time:

1. Radar Rainfall (measuring rainfall rates and total fall by radar).
2. Hurricane Detection and Study
3. Echo Flutter (analysis of the fluctuating character of weather echoes and its correlation with raindrop motion).
4. Snow Echoes and Polarization Effects (characteristics of precipitation echoes which are peculiar to snow and ice as compared with rain).
5. 'Scope Meteorology (the use of radar for studying the structure of storms and assisting the forecaster).
6. Airborne Radar for Weather Avoidance.

RADAR RAINFALL

"Radar Rainfall" divides naturally into two different problems. At M. I. T. we are now trying to tie down the actual relationship between the intensity of the radar return from a small volume of space and the raindrops in that space. The N. R. L. group, represented here by I. Katz, is trying to do the same with airborne tools and techniques. That is one particular problem which should be nailed down and the 6 or 7 db discrepancy accounted for. It is a very interesting problem. I have a hunch that the explanation lies somewhere in the meteorological part of the measurement. Solution of the problem may teach us something important. The second, somewhat different, problem is the area measurement of rainfall by radar. I was talking to Marshall about this last night and we hoped the group here in Illinois would concentrate on it. They have an unusual rain gage network, giving them a beautiful chance to work in that field. It would seem like a good division of effort for N. R. L. and M. I. T., with the help of other people, to concentrate on the point intensity measurement, and for the group in Illinois to concentrate on the correlation between their rain gages and radar intensities. From an engineering standpoint the important stunt is to be able to measure rainfall with a particular radar, calibrated against some other direct means such as a rain gage. The absolute value of the echo intensity is unimportant to the hydrologist.

HURRICANE DETECTION AND STUDY

This topic was covered very adequately in the preceding panel this morning. I do not think we need to spend any more time on that now. Mr. Jorgensen certainly has a dynamic and interesting program going on in Texas which seems only to require more help with gadgets and personnel during the hurricane season. Maybe some of us could take off from the more northern latitudes where there is not much "weather" in September and October and concentrate on helping him get the data, bringing some cameras, gadgets and such. This might be a practical way to get the information faster. You certainly have a dynamic group tackling the problem, Mr. Jorgensen.

ECHO FLUTTER

This is a short term for the complex study of the detailed nature of weather echoes. Walter Hitschfeld and Aaron Fleisher explained yesterday the relationships between the motions of individual raindrops and the variations in pulse-to-pulse echoes. These variations can be used to observe the effects of, and possibly to measure, winds aloft, raindrop fall velocities and turbulence.

One very exciting thing about this approach to turbulence did not get emphasized particularly yesterday. This method of measurement has a unique quality in the field. Most turbulence studies to date have been based on a few point measurements within a large volume of space. With radar, in this case, we get instead of just a few point measurements, billions of them simultaneously. Each individual raindrop is a recording point. The radar and the gadgets we are tying onto it then becomes an analytical machine which is analyzing the data from each one of these individual points. It may well prove to be a most valuable tool in turbulence studies.

SNOW ECHOES AND POLARIZATION EFFECTS

The Geophysics Research Division, of the McGill group and the Mt. Washington group, have a very dynamic and progressive program in this field.

SCOPE METEOROLOGY

This topic refers to the very important use of radar in weather forecasting. This has been one of our major assignments on the M.I.T. project, but I do not feel we have made much headway on it as yet. We have collected many data and watched many storms on our scopes. This work has educated us considerably and has advanced our knowledge of the structure of storms but it has not put us in a position to be of much help to the forecaster. What we might strive to do is to learn from our radars and flight observations enough about the structure of big cyclonic storms so that we can, from the conventional data, specify in much greater detail just what is going to happen within these cyclonic storms as they develop, where instability showers will form and so forth. Even when the motion of a storm is well forecast, we often miss forecasting the amount and placement of the precipitation. The radar should give us a better clue to the internal structure of the cyclone and what occurs in greater detail. I am sure there is much internal detail within general cyclonic areas that is not appreciated. Within a frontal storm that may be 500 to 800 miles in diameter there are small areas of convergence and instabilities that are, in effect, storms within a storm. There was a storm in New England about a month ago which was forecast to be only a few scattered showers, and moderate wind. In eastern Maine we had a big "southeaster" with heavy rain all day, yet it did not appear on the weather maps at all. It behaved like a disturbance along a front which had moved off the coast but was not picked up on any of the synoptic maps. Small but intense storms of that type may be very important.

AIRBORNE RADAR FOR WEATHER AVOIDANCE

F. C. WHITE. —One of the practical applications of radar that we might add to this topic, from the point of view of the people who fly, is the use of airborne radar (of the proper wave length) with the addition of isoecho contours. Preliminary data, obtained by American Airlines during work on the Navy contract, indicates that, by avoiding the areas where isoecho contours converge, a large portion of thunderstorm turbulence can be avoided. We should endorse the Navy program which is continuing the American Airline's work with airborne radar. The Navy program has two objectives: first, continued investigation of the contour convergence technique, to enable determination of whether or not isoecho contours can display the most turbulent portions of thunderstorms; and secondly, a determination of the deterioration of airborne radar display that results from thunderstorm precipitation. This second objective will be attained through the simultaneous use of both three- and ten-centimeter radar during thunderstorm probing flights.

DISCUSSION

BY

F. C. WHITE, D. M. A. JONES, D. ATLAS, I. KATZ, S. E. REYNOLDS, G. E. STOUT, P. M. AUSTIN, J. C. FREEMAN, H. R. BYERS, R. J. BOUCHER, L. J. BATTAN, G. E. DUNN, E. L. WILLIAMS, D. M. SWINGLE, J. R. ANDERSON, M.H. LIGDA, H. W. MAYNARD, W. B. GOULD, A. C. BEMIS

A. C. BEMIS. —I agree. Could we go back now to Radar Rainfall. Horace, do you have some comments to make?

H. R. BYERS. —Well, more or less along the lines of use for hydrologic purposes. Here in Illinois we of the State Water Survey are working for the taxpayers. We have told the taxpayer that we can not measure rain that occurs over a small basin with the existing rain gage network, even though the gages are more closely spaced in Illinois than in most states.

There is hardly a week passes, during the thunderstorm season in Illinois, but that somewhere in the state a small stream floods and serious damage is caused. Radar can do a great deal for measuring small storms in small basins. As a matter of fact, that is about the only place that radar is unique in rainfall measurements because for large storms and large basins the general rain gage network, while not adequate, is at least close to adequate. So we must concentrate, then, on small storms and small basins, and that is what is being done in the Goose Creek network that Mr. Jones told us about the other night.

At that discussion I pointed out that the time element is very important, and I think here at this meeting we have stressed a little too much the detail of getting rain measurement and have not considered the most important thing from the hydrologist's point of view, which radar gives us very obviously, and that is the duration of rainfall over a given spot. Suppose we have a storm for which the precipitation indicated is 3 inches per hour, and we have a little basin and the storm is drifting across the basin. If it moves across in half an hour you get an ordinary situation, an inch and one-half of rain. It may continue raining 3 inches per hour but it moves on across, depositing that rain at all the various stations through the state and nothing spectacular happens in the way of floods. On the other hand, if you have a system of cells, raining 3 inches per hour, that takes, say, three hours in getting across, then you have a real flood. Of course, that is something you can get very easily on radar. You can look at your radar and see that this storm is developing over that area and staying there. Of course, you have serious floods. We had one precisely like that in Chicago last week. It did not rain very hard but the storm was there for six hours.

Then the next question is: What about the large areas of rainfall? The hydrologist wants to know how much rain is falling over a large area. Some hydrologists will say if you take in a large enough area, say the whole state, then although the rain gages are quite far apart, you are getting a fairly good sample. Fellows like George Benton are making computations of the total moisture exchange that is taking place in the atmosphere—evaporation, precipitation, etc., trying to find out just what the hydrologic balance is. We are not sure—I am not sure he is either—that we know what the rainfall is over a large area. Well, radar, of course, can do that, for it not only samples as rain gages do but scans every square inch of area. What we need, then, is a future development of some means of totalizing the scan of the radar echo so we can then get some idea of the rainfall. Even if we were 100% off in these rainfall rates, we would still be doing better than we are with the rain gage networks. I would like to suggest the development of some means of masking off the radar scope successively for a group of different basins and then have some kind of photo-multiplier, storage cell, or something that will keep totalizing the echo that is received over each one of these basins over the entire range of the radar. Then you could have it arranged so it would ring a bell or something after the rain exceeded a certain value over any basin that would indicate a flood. That is something that could be done without too much trouble, and I hope that we can have some development like that in the near future.

A. C. BEMIS. —That surely sounds like the right plan of attack to me.

D. M. A. JONES. —I would like to bring up the point that here in Illinois we just recently have had a graphic illustration of the inadequacy of raindrop size distribution measurements. If you or somebody else does not do it, we will have to do it.

D. ATLAS. —Dr. Byers' suggestion for meas-

uring total fall over a watershed is well taken. Until now we have been neglecting our responsibility to the hydrometeorologist by failing to provide practical techniques for this purpose. Recent electronic developments, such as storage tubes, make echo totalization feasible both in space and time. But there is another hope for areal rainfall integration; that is the use of the otherwise bothersome attenuation at short wave lengths. Since attenuation has been shown to be linearly related to rainfall rate, the attenuation through a storm is proportional to the integral of rain intensity. In order to measure attenuation, use must be made of two wave lengths, one of which is not attenuated (i.e., 10 cm.).

I. KATZ. —One of the difficulties we have is that we are trying to measure rainfall distributions on the ground and looking with radar at areas aloft. To circumvent this difficulty either an airborne or balloonborne equipment should be developed which will give us good drop size information. As I understand it, Bob Cunningham and I are the only two people in the United States working on an airborne disdrometer or anything like it. I may be wrong, but I think that is true. There is an uneven distribution of emphasis here; most people are working directly or indirectly on radar but very few people are working on meteorological instrumentation to get the important atmospheric parameters we need to accompany the radar data.

A. M. BEMIS. —There are a number working on drop sizes at the ground.

I. KATZ. —Yes, but we must obtain drop size information in the regions aloft from which the radar echo originates.

S. E. REYNOLDS. —It seems to me that we might mention some other classifications here; one is techniques for the use and application of radar in cloud physics, particularly those relating to precipitation mechanisms. It seems to us that one of the most important times to study these is at the time of the initial radar return.

There is one problem that is bothering us considerably now. That is the discrepancy between Thunderstorm Project and our data relating to the height at which the initial radar return occurs. As most of you remember, the Thunderstorm Project data show that it occurs somewhere a little warmer than -5° C., whereas ours show that it occurs around -15° C. It seems to me that the important point here is, if the Project data are right (and in my opinion they are much better than ours because they were taken with much better instrumentation), it almost precludes the initiation of precipitation in their storms from an ice mechanism, whereas our returns occurring first at -15 at least very strongly suggest that it did start in accordance with the classical Bergeron theory in our storms. I think a little further study on this point, which we hope to do fairly soon, might clear it up. I think it is very important to find out if precipitation in the Midwest begins in the warm regions of clouds and in the Southwest begins around -15.

UNIDENTIFIED.[1]—(A statement to the effect that the particles might have fallen to the observed position.)

S. E. REYNOLDS. —I do not think that will work in the case of a thunderstorm because (while the Thunderstorm Project did not take photographs of the clouds) our data show that the clouds are going up at the rate of around 1000 feet per minute for 4 or 5 minutes before the radar echo appears, which means that these things could not have fallen back down in that length of time.

UNIDENTIFIED. —(Apparently a statement to the effect that the circulation is complex.)

S. E. REYNOLDS. —Well, it happens that this is another thing which we can see very well out there in our clouds. Individual updrafts affect limited regions of the clouds; that is, the part which is going up is not several miles in diameter—it is perhaps several thousand feet. The current structure in the region in which the initial radar return appears seems a quite simple thing. It is simply going up.

A. C. BEMIS. —It seems to me this brief discussion points out that here in cloud physics, as in all the rest of the meteorology, we find each storm is different from the one before. It is not going to be sufficient to get one beautiful, complete set of measurements on one storm. We cannot make our measurements just once, but must repeat them a great many times in different parts of the world. This makes Katz' comment that we need more instrumentation even more significant.

G. E. STOUT. —I should like to add at this time that if at any time anyone wishes to use the data from the rain gage network you are welcome. Furthermore, if anyone desires to coordinate flight observations with rain gage network data, and surface observations with rain gage network data and surface radar photos, we would be very happy to cooperate.

D. ATLAS. —I feel that we should come to some conclusion with regard to the present state of knowl-

[1]During the discussion there were comments by persons who were unidentified in the recording.

edge of raindrop size distribution and the course which further research on this problem should take. It is my personal opinion that we have gone about as far as possible in determining the correlation of summation d^6 to rain intensity. At least a half dozen independent studies have shown a useful correlation of these parameters with standard errors of estimate ranging from about 20% to 40%. This is undoubtedly due to the variations in rainfall characteristics within a storm and from storm to storm. When we consider the various conditions under which precipitation elements are generated and the many ways in which they may be modified in falling to the ground, it is a wonder that the variability is not greater. Let us admit that this is the inherent limit and work on reducing the errors with which the radar can measure summation d^6. One of the practical problems in this regard is the determination of the range to which radar observations can yield useful measures of rain intensity in the face of increased beam dimensions, sampling altitude, and attenuation.

Lest my statement be misinterpreted, I would like to add that there is much to learn about drop size distribution and its relation to the generation and growth mechanism. It is that further study is necessary. Perhaps an understanding of the basic physics of size distribution will provide useful guides for the use of radar-rain correlations.

P. M. AUSTIN. —It may well be that we have gone as far as is feasible in attempting to obtain a general relationship between rainfall rate and radar signal intensity. I believe, however, that a more accurate or detailed relationship must be established if we wish to study quantitatively the details of storm structure which have been observed, in a qualitative manner, on the radar scopes. In order to do this, the drop size, rainfall rate and radar signal intensity studies should be continued, but with more emphasis on the behavior of these variables within any one and the differences or similarities from one storm to another. Also, if there are any geographical differences in the composition of rain, this should be taken into consideration.

D. M. A. JONES. —Is this not saying in another way that each radar set must be calibrated as an instrument by itself?

P. M. AUSTIN. —I think that you cannot rely on the radar parameters for accurate measurement. Probably a standard target should be used, or you might calibrate the radar against a rain gage.

J. C. FREEMAN. —As an uneducated outside observer, it seems to me that it is possible to calibrate the radar set locally every day there is rain and at least get some measure of the precipitation all over the scope.

H. R. BYERS. —I should like to make one statement before we go any further. In spite of what my own colleagues say here in Illinois, I think that the difference in rainfall between Boston and Champaign is overemphasized.

R. J. BOUCHER. —Drop size distributions from all the data worked up to date show that there are no essential differences in rainfall with location. We have data from Canada, Cambridge, Washington, and Hawaii, and all of them agree quite closely. We would not expect any differences due to locality.

D. M. A. JONES. —I think that Dr. Byers misinterpreted my statement somewhat. We are working with thunderstorms, and since most of our rain is derived from that, it may be that we have more thunderstorms than do other regions.

J. C. FREEMAN. —Here is an important practical question about this radar rainfall. Does this conference recommend that every group interested in hydrology get itself a radar set with which to measure precipitation?

A. C. BEMIS. —It seems to me that the discussion here might serve to recommend such action.

J. C. FREEMAN. —I want this made clear: Is now the time to start doing it everywhere?

A. C. BEMIS. —I would. It appears the Illinois group is having good success.

H. R. BYERS. —I think every organization seriously concerned with the problem of hydrology and rainfall measurement should enter this field. I must say that Dr. Buswell has done a remarkable job of developing the State Water Survey to a level of scientific achievement in this regard not approached elsewhere.

L. J. BATTAN. —If one uses the radar echoes just to draw the isohyetals to fill in the spaces between the rain gages, I think that is getting some use out of it.

A. C. BEMIS. —Could we go on to another topic? I would like to hear more about the airborne program, particularly the turbulence part of it, which might be a combination of "Echo Flutters" and "Airborne Radar for Weather Avoidance."

F. C. WHITE. —Some may not be familiar with the work that was done by American Airlines during 1947, 1948, and 1949 under contract to the De-

partment of the Navy, Bureau of Aeronautics. During the summer of 1950 similar work was accomplished by the Naval Air Station, Patuxent River, under the direction of the Bureau of Aeronautics. Both of these programs included flight testing of a special circuitry attached to three-centimeter airborne radar which displays thunderstorm precipitation. The circuitry creates two isoecho contours, a threshold contour, and an erasure contour, separated by some ten decibels of signal level. The effective gust velocity (a measure of turbulence resulting from thunderstorm gustiness) was recorded by the National Advisory Committee for Aeronautics (NACA). Through analysis of pictures of the three-centimeter radar PPI display, a correlation between squeeze of the contours and turbulence was found to exist. Where the contours were close together (the change in rainfall rate per mile was considerable), more turbulence and stronger gusts were encountered. These tentative conclusions were based on limited data, some 600 miles of flying. It should be recalled that the data were analyzed at approximately 1/2 mile range to eliminate the effect of signal attenuation on the display of contours. The Navy is in the process of establishing an airborne radar project at Naval Air Station, Patuxent River, to continue this flight testing and to attempt to obtain further correlation of the relationship between contour separation and turbulence. The Navy also will determine the exact effect of attenuation of the three-centimeter signal by taking simultaneous data using a ten-centimeter airborne radar system. The Navy plans to use a PB1 (B-17G) aircraft which should be flying by the spring of 1952.

A. C. BEMIS. —I would like to emphasize one point in connection with the use of radar for studying the structure of storms. It is most important to make constant use of vertical sections through the atmosphere such as are given by an RHI. Cunningham, in his precipitation physics studies, has always used our RHI records much more than PPI. Since the information of precipitation requires vertical motion, it is obvious that vertical sections are valuable; yet we still lean too heavily on the PPI. One reason is that few radars are equipped for RHI scan, but I fear another reason is that it is too easy to look at a PPI. The PPI is such an attractive thing to just sit and gaze at. There it is, neatly plotted for you and easy to understand. You know just where it is raining. However, if we are really going to get into the business and learn how the rain is formed and where it is coming from, we need vertical information. Actually, it is not hard to get. Bowen, in Australia, simply turned a PPI radar on its side and immediately got very valuable information.

I am looking forward keenly to the use of more cloud base and top radars as bringing a tremendous advance in this general cloud physics field. There has been too little tie-in in the past in radar work between the precipitation and the clouds from which it comes. With these cloud sets, as Mr. Gould showed you yesterday, you have the relationship beautifully presented. You can observe the thickening cloud deck overhead and actually see the precipitation begin to fall out of it. The Geophysical Research Division at Watertown also has a vertical pointing set to detect clouds. We are now getting some real data in this field, and they should be very important. I hope more and more investigators will come to the use of vertical scans whether they get the information from a vertical pointing cloud set or some scanning type such as the TPS-10.

Marshall made an interesting comment on this last night. He said they had taken their TPS-10 down to the weather station in Montreal. Marshall's group was not using it very much, and the weather boys were very keen about it and used it a lot. They had planned to rig it up for PPI scan but just had not gotten around to it. Marshall thought that this was probably a good thing, forcing the weather bureau people to use the RHI. He found them more enthusiastic about radar than most weather bureau groups who had been using PPI. As far as forecasting goes, routine use of a PPI adds little to what one learns from the teletype circuits.

We would like to hear about the Weather Bureau's plans for the increasing use of radar. We know you are using it quite extensively now but would like to know more about your plans for the future, Mr. Dunn.

G. E. DUNN.* —There is a map on the wall in the room to our left, and I also have one here, which shows current and proposed radar installations. You can see that we will have very good coverage in the tornado and squall line section of the Great Plains of the Middle West. Also in the hurricane-susceptible area along the Gulf of Mexico and south Atlantic Coast, coverage will be ample. Still another project, now underway, is the establishment of radar installations for storm detection around the major cities of the United States, such as Washington, New York, and Chicago. When present plans are completed, about 60 percent of the United States, especially the eastern two-thirds of the country, will possess a fairly good storm detection network.

UNIDENTIFIED. —(A question in regard to the possibility of cooperative installations by the United States Government and private industry.)

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G. E. DUNN. —I believe M. I. T. has a radar installation, and it would seem that some cooperative agreement could be worked out which would provide the Boston Forecast Center with the information which the latter could use in its local forecast services. At the present time radars are installed and in operation in New York, Washington, Miami, Tampa (the latter in cooperation with the University of Florida); Burrwood, Louisiana; Brownsville, Texas; Wichita Falls, Texas; Wichita, Kansas; and Norfolk, Nebraska. Cooperative agreements have been worked out with private companies at several locations--the Dow Chemical Company at Freeport, Texas is one. Another at Grand Isle, Louisiana has been discontinued. Similar installations are located at Gainesville, Florida, in connection with the University of Florida, and another at Victoria, Texas, with the Copano Research Foundation. Cooperative arrangements of one sort or another between the Weather Bureau and private industry are in effect at Corpus Christi, with the Central Power and Light Company, and the one here at El Paso and Champaign with the State Water Survey Division. Plans are underway for a radar installation at the Weather Bureau Airport Station in Chicago in cooperation with the Commonwealth Edison Company. Most of these installations are the APS-2F. During the next year or two Weather Bureau installations are planned, budget permitting, at Amarillo, Texas; Charleston, South Carolina; Hatteras, North Carolina; Topeka; St. Louis; Philadelphia; Boston; Detroit; Buffalo; Goodland, Kansas; Tulsa; Des Moines; Dodge City; and Springfield, Missouri.

In connection with hurricane detection it is expected that the Navy at Jacksonville, Florida, will shortly install a radar there or perhaps has already. There is another radar installation at a base near Melborne, Florida, and two or three additional in the Bahama Islands are contemplated in the next year or two, as well as several in Cuba. From the standpoint of hurricane forecasting in the southeastern United States, the entire coast line will be covered quite well from Cape Hatteras around to Brownsville and extending seaward over a large portion of the Gulf of Mexico and southeastward as far as San Juan and much of the northern Caribbean. Thus it seems fair to say that already, or at least within the next few years, radar is a very useful tool in hurricane forecasting.

UNIDENTIFIED. —Are all your installations APS-2's?

G. E. DUNN. —I believe we have that equipment at every Weather Bureau station except New York, which has an APQ-13.

A. C. BEMIS. —Another radar coming up in a few years is the new CPS-9, as soon as Ed Williams gets them built. Don, do you have any remarks to make about your proposed network for the siting of the CPS-9's?

D. M. SWINGLE. —Siting the CPS-9 is something that I have nothing to do with directly, but the responsibility is with the people who are really buying them, the Air Weather Service.

G. E. STOUT. —I might add a little bit about it. They tell me that one of the first sets will go in at Chanute Field so that they can train operators and maintenance personnel. They are also planning a ring of stations in the Caribbean area for hurricane work.

UNIDENTIFIED. —How many CPS-9 sets are on order?

E. L. WILLIAMS. —Forty-eight. The Navy is getting five.

D. M. SWINGLE. —There is just one thing I want to mention about getting the information from the APQ-13, that is the difficulty in respect to maintenance due to military routine, etc. and a great deal of difficulty in their siting. Now, the APQ-13 network may not be a first class network, totally independent of the qualities of the set. It is an unfortunate situation since the APQ-13 in itself is capable of doing a lot more for us than it is doing.

A. C. BEMIS. —I would very much like to hear some helpful suggestions on the problem of the use of radar for synoptic purposes. We have had it in mind for a long time, but we have not made any concrete headway on it as yet.

J. C. FREEMAN. —The hourly reporting stations are too far apart to help us find synoptic models of what we see in the radar scope. However, at many of these stations continuous records of pressure, temperature and wind are made. These continuous records can help fill in the large blank spaces between the stations for slowly changing patterns.

Several times during the conference I have emphasized the need for study of the radar scope by synoptic meteorologists whose primary interest is in the weather. Such men can see things that a meteorologist whose primary interest is radar or cloud physics will pass over.

For example, when Tepper was studying squall lines and saw time-lapse movies of those across the cloud physics nets, it was obvious to him that some "line" was coming across the scope and starting the formation of thunderstorm cells. This observation has also been made by H. Maynard here at Urbana and by other weather men.

A. C. BEMIS. —The trouble with most of us is that we tend to use our radars for our own edification and amusement instead of making a real effort to analyze what we see, classify it, and then get it down on paper for the benefit of others. The time has arrived when a textbook or operations manual must be written.

UNIDENTIFIED. —Are there not two different synoptic codes floating around just to complicate things in transmitting data?

A. C. BEMIS. —Well, codes are a big problem, but first we must decide what information is to be transmitted.

H. R. BYERS. —I might point out that in the Thunderstorm Project there has been quite a study made of squall lines. If you remember, there was not too much in the chapter of our report on the subject. In connection with that, Captain Pope of the Air Weather Service made a study which I think was put out as some sort of Air Weather Study (Restricted, unfortunately), in which he took observations of squall lines and thunderstorms from the hourly teletype reports compared with what you see by radar. I cannot quote directly, but there was one day in which there was more than the usual number of thunderstorms over the region of Chicago, Great Lakes to Pittsburgh and down to Tennessee, covered by our range on the CPS-6. On that day not a single hourly weather station reported a thunderstorm. Some did report distant lightning.

I should also like to ask Mr. Dunn of the Weather Bureau if it is not true that you make great use of reports in regard to squall lines.

G. E. DUNN. —We make considerable use of the reports coming to us from El Paso, Illinois. However, under present procedures for the Service "A" teletype circuits, C.A.A. is not required to relay rareps from one circuit to another so that if a forecasting center is not on the same primary circuit with the radar station, receipt of the radar information is quite irregular, to say the least.

J. R. ANDERSON. —Wasting of weather information, the way in which it is taken down at individual stations, is pitiful from a record standpoint, to put it brutally, as far as utilizing it for research purposes is concerned. There is nothing I have seen to be extracted from this data in a usable form. A method such as photographs should be inaugurated for record purposes. I have seen no pressure applied to have photographs made at all these places. It is a very simple process to do it. I have seen no system of taking synoptic weather information off of the scope in code form that I thought came anywhere near approaching the problem of record information in a usable form, say to Mr. Dunn.

D. ATLAS. —I feel at this time that we shall not be able to fully utilize the available data unless there is some manner of transmitting the radar pictures to a central collecting point where they are integrated into a comprehensive map of precipitation echoes. There is so much information in the character of the echoes which cannot be coded, unless, of course, we have an echo catalogue. As for techniques, there are a number of possible ways of transmitting and combining the scope displays for re-transmission. These might make use of existing coaxial and microwave circuits, although I have no idea of the expense.

UNIDENTIFIED. —What type of communication network would you use? Would you have some special type of machine at each station?

D. ATLAS. —No, I think you could do this with the present facsimile machine if the video data are transmitted to a central point for re-transmission on the fax network.

G. E. STOUT. —I think that the whole communication problem has to be settled by some governmental agency. To me looking on from the outside, and not condemning the Weather Bureau, it appears that they are a little slow realizing the problems in jumping on the ball and getting things moving. The Water Survey volunteered to send in rareps, and we found that there were two codes, the Air Weather Service code, and a Weather Bureau code. Furthermore, there may be another code. I have seen some rarep reports that I could not decipher. A group at Memphis uses another code which is a combination of the two codes. As far as the plotting goes, we were very successful in giving radar reports to a local radio station. This was discontinued as we moved the radar station from El Paso to Champaign.

UNIDENTIFIED. —You can put it on a television station when you get set up here.

G. E. STOUT. —That is right; we just made a copy of what was showing on the scope and relayed it to the radio station in a layman's language.

UNIDENTIFIED. —Mr. Jorgensen brought up the point of using the longitude and latitude rather than the range and azimuth for radar stations. I should think it would save quite a bit of time in the Weather Bureau station in putting the scope information down in a usable form.

UNIDENTIFIED. —Possibly, but you all know that land lines are extremely crowded. It appears

to me offhand that you may need latitude and longitude to degrees, minutes, and seconds, to get sufficient detail.

D. SWINGLE. —We have the know-how to transmit all the stuff by video to a central collection agency. We have yet to find the man with the dollars. It is quite expensive. There are other schemes for getting it through cheaper. These are the things that have to be threshed out first.

UNIDENTIFIED. —I was once a forecaster, and it seemed that there was much stuff being transmitted that was worthless for forecasting purposes. I should want to see, before we go any further, a critical survey of what value the information will have to a forecaster. I wonder whether we cannot eliminate some of the information we now have on the teletype.

A. C. BEMIS. —Is it not true that, in general, the forecaster has much more information than he has time to use, some of which may be valuable and some which may not?

G. E. DUNN. —Yes, but you will find that the River forecasters and Flood Control people will want river stages and rainfall reports, and the Climatologists will want crop reports. There is certain other information transmitted on the circuits which has little to do with forecasting and certainly the competition for circuit time is quite severe.

E. L. WILLIAMS. —This whole problem of transmitting information from the radar has been discussed over and over and is of great importance. I would like to ask a question. Is there any agency now actively working on this problem? Second, what agency is responsible for it? Third, if there is none and it is not going on, how does it get started so that something can be done about the problem? There are going to be 48 CPS-9's going out into the field, and what is going to be done with the information from them? How is it going to be used on top of all the other meteorological information being transmitted?

H. R. BYERS. —A lot of people could, probably, answer that better than I, but I will tackle it. The whole transmission system is decided on in the first place by an international commission. Secondly, there is the joint committee of the Weather Bureau, Air Force, and Navy; they work very closely together, and I think they are rather conscientious about it. But it is only partially their responsibility. The people in communications also must discover ways of getting information transmitted. If you have ever attended an international convention on communications, you will appreciate some of the problems.

A. C. BEMIS. —I feel you are neglecting problem number one. Our major assignment in all this is getting the basic data ready to transmit. Of course, it is hard to do this until you know the capacity of the transmission channel which is to be used; but I still feel we are a long way from knowing what information we should take from the source. We understand the simple problem of transmitting PPI information on a squall line or cold front or any other sharply defined region of precipitation that a person can place accurately on a map. But there are many other characteristics of precipitation echoes that may be significant yet much more difficult to interpret and certainly more difficult to describe.

R. J. BOUCHER. —Since there seems to be no one project assigned to the problem of fully exploiting the radar as a tool of synoptic meteorology, I would like to suggest the organization of such a project whose sole purpose would be to devise techniques for obtaining from the radar the most useful information for meteorological purposes.

A. C. BEMIS. —Mr. Gould assigned that project to us at M.I.T. quite some time ago, but we have not made much headway with it as yet. I would appreciate suggestions as to how we might attack it more effectively.

M. H. LIGDA. —One of the chief difficulties is that radar essentially tells you one thing and that is where it is raining. It does tell you this, but it does not tell you the wind velocity, temperature, and other data which are useful. Because of this fact the forecaster, the analyst, and the physical meteorologist have not yet been able to establish definite or clear-cut relations between radar indications and synoptic conditions. Rareps, as such, are of very little general use to the continental forecasters at present. That is one of the main reasons the present work is progressing so slowly. It is primarily a problem of interpretation, and I have a feeling that this whole business is drifting of its own accord towards the conventional ideas of data transmission. I am a little bit afraid of this whole idea of getting, for instance, a lot of fancy fax equipment going and providing every weather base with the precipitation pattern over the whole United States. It is going to be an awful looking mess for one thing. The second thing is that, to be frank, we do not know just what it means yet. I believe that it is going to be necessary to think more in terms of typing or indexing the PPI scope in terms of such quantities as the cell life, duration, and vertical extent of cells—work which was started by such people as Mather and McDowell, but needs

further extension. Looking at it more abstractly, we might compare our present situation to the early days of aerological ascents. Isolated observations were available but interpretation of the sounding in terms of stability air mass type, and other parameters so vital to present day forecasting, was not possible. It is just at that phase of the game where we now find ourselves. It is very much the same type of problem; the conclusions are obvious.

D. SWINGLE. —The Signal Corps and the Air Weather Service have been talking over a joint study to determine just how much information is needed for weather purposes. In particular, we have been considering some sort of a limited network as an experimental proposition in order to find out how far we want to go in this business of transmission of data. For instance, on the CPS-9 we have range resolution of about half a mile, while at 100 miles the azimuth resolution is only about three miles. For the general use of the data there is probably no point in directly transmitting the data to better than three miles. This is the sort of thing we hope to uncover. If we discover that all we need to know is whether there is a rain echo within a given ten-square-mile area, then data transmission will become a relatively simple problem. If we want further information, the problem will be more difficult. These are the things we have to look into and find out how much we have to transmit.

A. C. BEMIS. —I certainly hope we can use more radar information than just where it is raining. Radar can give us much additional meteorological information.

UNIDENTIFIED. —How much information do you want? You and I look at it and observe a storm moving at 25 miles per hour. The RHI shows that it is developing vertically. But how much of this information do you want to send? To what extent do you want to pre-interpret the data? To what extent do you want to transmit every detail? These are the things we want to decide before we set up any sort of a network. As Herb says, "Just exactly what information do we want?"

UNIDENTIFIED. —What do you mean by "classifying pictures"? Do you mean that you hope to be able to pick out on the scope certain models, so that, for example, if you have a squall line, your scope appearance would match some picture in your classification book?

S. E. REYNOLDS. —It seems to me we might come a lot closer to picking out general types. It is going to be a long time before you get a particular synoptic situation repeated.

UNIDENTIFIED. —The difficulty with RHI is that you get vertical sections through the atmosphere, yet if you try to transmit volume information the task is enormous.

S. E. REYNOLDS. —It might be easier to start with the PPI and get a synoptic picture out of the PPI.

M. H. LIGDA. —Well, anybody who has had quite a bit of experience looking at radar scopes gets at least a vague idea of what the weather is like without reference to a synoptic map. You get a pretty good impression of how long the precipitation is going to last (which is just about as good as the conventional New England forecasting), subject to coast line effects, etc. You get a pretty good idea from looking at the PPI scope whether the precipitation is due to convective or warm frontal action. Rain and snow have their own characteristic appearance on the PPI scope. This is just an accumulation of experience, and it certainly can be systematized in such a manner that can be transmitted to other persons up the line who are going to have the stuff hit them in another eight or nine hours and tell them what to expect.

H. W. MAYNARD.* —The trouble with trying to line up different classifications is that they may run into the thousands.

M. H. LIGDA. —I do not believe that there will be that many classifications. I think it would be more like fifty.

A. C. BEMIS. —I want to ask Mr. Gould a question about the base and top radar. We were speaking a while ago of the terrific importance of vertical sections and the valuable information that one gets from a "cloud set." I believe there are only two such radars that are now operating, yours at Belmar and Atlas' machine at Watertown. Is there any chance of getting continuous weather data from these two sets so that we can attempt to analyze it, using the same procedure that we now use for joint observations with our two CPS-9's? Can we begin to make synoptic studies with the cloud set? Is that going on already?

W. B. GOULD. —No, that has not been going on. I think something is needed along these lines. I have learned today that Dave has similar equipment working in Watertown. It would be possible, I should think, to take day-by-day records from the two sets and compare them, and possibly we might then be able to gain some experience. We have not been running our set regularly, but we should start in doing it on a regular basis in a month or so.

A. C. BEMIS. —Good. I think the weather people there in the Met section would be keen to jump onto that program and start correlating it with the synoptic situation.

G. E. STOUT. —I believe you can classify these PPI pictures into various types like we do the synoptic weather map, like a cold front, etc. You can break them down into a series of types, and not too many different cases either. We have observed a couple of cases which we think are associated with tornado situations. I think it is possible to classify them and not have ten thousand classifications either.

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STATE OF ILLINOIS

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1952-1955 ILLINOIS DROUGHT

with special reference to

IMPOUNDING RESERVOIR DESIGN

BY

H. E. Hudson, Jr. and W. J. Roberts

DEPARTMENT OF REGISTRATION AND EDUCATION

VERA M. BINKS, *Director*

STATE WATER SURVEY DIVISION

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SUMMARY

A large part of central and southern Illinois experienced a severe drought beginning early in 1952. By June 1955 the drought situation had greatly improved. Water shortages occurred in the fall of 1952 as a result of rainfall deficiencies. The situation was partially relieved by above-normal rainfall during the first three months of 1953. Rainfall deficiencies began to grow again after April 1953. By September 1953 it was apparent that the effect of the 1952 rainfall deficiency had not been fully overcome. The situation grew worse and by January 1954 there were 41 Illinois communities with serious water problems.

In January 1954 a program was set up by the Water Survey to report the drought situation monthly. Data were obtained on water supplies through visits, correspondence and telephone communications with communities affected by the drought. This program was continued until January 1955. By June 1955, all drought-connected water shortages had ended. Figure 50 shows locations of places mentioned.

Sources of Water Supply

To appreciate the drought, it is necessary to have an understanding of the normal water supply situations in Illinois. Public water supplies are most frequently obtained from groundwater sources in the northern part of Illinois. In the southern part of the State glacial deposits are much thinner and their effect upon the storage and travel of water underground less important. The groundwater contribution to the low flow of streams is consequently lower.

Municipal water supplies depending upon surface water are most common in the southern portion of the State where groundwater is generally more difficult to develop. Cities in southern Illinois located on major streams take water directly from them. Other cities generally provide an impounding reservoir on a smaller stream, or a side-channel reservoir into which water is pumped from a main stream.

In the drought region of the state 202 public water supplies obtain water from groundwater sources. Ninety-eight systems in the region utilize surface water.

The majority of the impounding reservoirs in the drought region have been constructed since 1920. Eighty per cent of them were built prior to 1940. Ninety per cent of the reservoirs utilize drainage areas of less than 20 square miles. Median runoff required to fill these reservoirs is 3.3 inches.

Normal annual rainfall in Illinois varies from 34 inches in the northern part of the State to 46 inches in the extreme southern tip.

Average annual runoff in Illinois displays a marked geographic variation within the State. Runoff varies from less than 8 inches in the west and northeast portions of the State to more than 15 inches in the area south of the Shawnee Hills. These results are indicated by the analysis of 25 of the 165 gaging stations in the state.

Quantitative indications can be obtained of movement of water in the hydrologic cycle by comparison of rainfall, runoff, transpiration and evaporation, and water levels in shallow wells. From April 1 until mid-September vegetative and evaporative demands exceed the water available. This difference is made up either by water taken from the soil or by the reduction of transpiration and evaporation. From mid-September until the end of March, in the average year, water returns to storage in the ground. From the behavior of groundwater hydrographs for shallow wells which commonly show no particular recovery before the end of November, it is judged that the excess water available during September-November largely goes to make up deficiencies in capillary moisture. The groundwater level recovers between December 1 and April 1.

Vegetative and evaporative demands are so great that below-normal rainfall through July and August leads to recession of groundwater levels, reduction of soil moisture, and delay in recovery of streamflow. Such a situation occurred during the summer periods of 1952-55 drought.

Municipal Water Supply Difficulties

Total groundwater pumpage for municipal supplies in Illinois increased from 40 million gallons daily in 1900 to about 106 mgd in 1940 and to 135 mgd by 1948.

Of the 41 communities that faced possible water shortages in January 1954 all but 8 experienced population increases for the period 1930 to 1950. In 24 instances the increases were less than 20 per cent. The highest population increase was 57 per cent. Municipal pumpage in 24 of these communities increased by more than 100 per cent from 1938 to 1953. In three cases pumpage increased 600 per cent or more.

Deficiencies in hydrologic data appear to have had an important role in the failures of public water supply sources. At the time of design of most of the reservoirs now in use there were few streamflow data available. .

Twenty-two of the 202 public water supplies in the drought region relying on groundwater sources

experienced shortages. Of the 98 systems in the region that utilize surface water, 53 suffered shortages. Fifty-eight of the surface water systems in the region rely on impounding reservoirs, and 40 of these suffered shortages.

During the drought it became apparent that grave water shortages occurred most frequently with impounding reservoir supplies. This report therefore emphasizes hydrologic information directly applicable to design and appraisal of impounding reservoir sources of supply.

Of the 75 Illinois communities suffering water shortages during the drought, 38 experienced difficulty because developed source capacity was insufficient to meet demands, 28 had trouble because of unexpectedly large increases in demand, and 9 encountered difficulty because of declining capacity of facilities.

Extent and Intensity of the Drought

The area covered by the drought was generally within the southern half of the State although it extended northward into Hancock and McDonough Counties and north of Champaign County at times.

Maximum rainfall deficiency in the drought region of the State for the period April to December 1952 amounted to about five inches. Deficiencies in the drought region range from 10 to 15 inches during 1953 and from 5 to 10 inches during 1954. These deficiencies are depicted in map form in the report.

Maps of runoff during the drought are presented. These are based on streamflow records of the U. S. Geological Survey at 15 stations in the drought region. The patterns of runoff were generally similar to those of precipitation. For the 12-month period ending June 30, 1954, runoff in the area of Pike, Scott and Greene counties was 0.2 inches or less as compared with a normal streamflow of approximately nine inches per year for this area.

A procedure for analysis of rainfall and runoff data to determine the frequency of recurrence of drought periods of various durations has been developed. Using this procedure the frequencies of occurrence of low flows of various durations were worked out for five drainage basins in the southern half of the State for which records of 34 to 45 years duration were available. Similar frequency determinations were made for five rain-gaging stations in the same area having 48 years of record.

On the basis of streamflow data for the period 1914-1955 the drought in 1952-54 appears to have been a severity that would occur once in 75 to more than 100 years. The driest 36-month period (ending June 1955) appears to have been a severity that would occur once in approximately 100 years. Rainfall data indicate the drought to have been of a severity that would occur once in 60 to 100 years. The most representative value available for evaluating the drought as a single event was the median recurrence interval. On this basis the drought of 1952-55 appears to have been a severity that would occur once in 83 years.

Estimating Reservoir Storage Requirements

The report reviews the methods available for calculating the amount of storage necessary for an adequate surface water supply. In general, these methods do not make possible the calculation of the amount of storage required to meet droughts of various durations and frequencies. The available methods do not provide a direct approach to the evaluation of evaporation losses from reservoirs. Most of the early reservoirs in Illinois were designed without allowance for loss of storage due to reservoir silting.

A new procedure, utilizing the streamflow frequency determinations discussed above is proposed for the determination of impounding reservoir capacity. The procedure may be divided into two parts: (1) determination of storage required for various draft rates and drought frequencies, and (2) evaluation of evaporative and sedimentation losses. For the study of draft rates, a weighing of low-flows of various frequencies and durations leads to the determination of the most critical storage requirement and drought duration for any given draft rate. These values have been consolidated into a single graph which enables rapid determination of the storage required at a given site for any drought frequency and draft rate in southern Illinois.

The data available on reservoir sedimentation in Illinois are summarized graphically in a way that enables the rapid estimation of damage rates.

The treatment of evaporative losses makes use of data on evaporation from lakes, which are reworked in such a way as to establish the most critical values resulting from the seasonal variations in evaporation rates. The report includes a study of the lake surface area exposed to evaporation and concludes that this value may be approximated by applying evaporation loss data to an area which is 64 per cent of the area of the lake when full.

A method of correcting gross evaporative losses by subtracting from them the amount of rainfall falling on the water surface is presented. This method takes into account the frequency of periods of low rainfall of various durations. These net evaporative rate losses are then applied to the effective area of the lake.

The effective surface area may readily be estimated by assuming a value of average depth for the lake. The median of the average depths of 41 reservoirs studied was 8.7 feet, but the use of an average depth figure of 10 to 15 feet is recommended, depending on location and reservoir size.

After combining the various factors, evaporative losses have been expressed in per cent of mean annual streamflow. So expressed, these losses are called net evaporative draft rates. A correlation between net evaporative draft rate and storage volume required was found. This correlation indicates that for a 10-foot average depth: at low values of storage, as for example a reservoir storing 50 per cent of one year's streamflow, the net evaporative draft rate would be 5 per cent of the mean streamflow, while for a reservoir storing three years' streamflow, net evaporative draft rate would be 15 per cent of the mean streamflow. These net evaporative draft rates appeared to be substantially independent of low-flow frequencies.

Values of net evaporative draft rate may be subtracted from the draft rate mentioned earlier in the study to produce values of net yield from reservoirs. The net yield is the quantity of water which may be taken from the reservoir for use, expressed as per cent of the mean stream-flow.

A graph showing the relationship of net yield to storage required for various low-flow frequencies has been prepared. This graph covers draft rates up to 80 per cent of the mean streamflow, storage values up to 300 per cent of the annual discharge, and low-flow frequencies having recurrence intervals ranging from 4 to 100 years.

It is suggested that the results of this study may be used for preliminary estimates of reservoir storage requirements and yields, and that the method developed may be applicable to local data for detail design purposes.

1952-55 ILLINOIS DROUGHT WITH SPECIAL REFERENCE TO IMPOUNDING RESERVOIR DESIGN

H.E. Hudson, Jr., Head, and W.J. Roberts, Associate Engineer, Engineering Subdivision.

INTRODUCTION

A large part of central and southern Illinois experienced a severe drought beginning early in 1952. By the spring of 1955 the drought situation had greatly improved but had not ended. In comparison with experiences during the 1930-31[1] and the 1940-41 droughts, a high incidence of water supply shortages became apparent. As the drought progressed, it became evident that a disproportionately large number of problems appeared to occur at communities relying on stored surface water sources of supply.

To orient the reader this report includes a section that outlines the history of the hydrologic and water supply deficiencies that arose during 1952-55 in Illinois. This history is followed by a brief description of the work needed to keep track of the situation.

For further orientation, a summary of the normal situation is included. This is followed by an analysis of the water-supply problems that arose and of the hydrologic characteristics of the drought. Data assembled in studying the drought proved useful for design studies and the report concludes with the presentation of the application of these data to impounding reservoir design.

Development of Drought Situation

Rainfall deficiencies commencing in April 1952 produced some water shortages by the fall of 1952. Supply shortage difficulties were reported at such widely separated locations as Marion and Pittsfield in southern and western Illinois. However, above-normal rainfall occurred in the first three months of 1953 throughout much of Illinois, raising water-supply reservoir levels, although in many cases, the reservoirs were not filled by these rains. The waterworks operator at Virginia, in west central Illinois, for example, reported that the spring of 1953 was the first spring since the reservoir was completed in 1933 in which it had not filled to spillway level.

Deficiencies in precipitation began to increase again after April 1953. By September 1953 it was apparent that the effects of deficient precipitation in 1952 had not been fully overcome by the above-normal precipitation in early 1953; and municipal water supply problems were much more numerous than in 1952. A survey in September 1953 indicated that 64 cities in Illinois were suffering some water shortages as evidenced by official restrictions of use. In 32 of the 64 communities, the shortages were attributed to deficiencies in distributive or treatment facilities. No field surveys of these cities were made by Water Survey personnel at this time, however, and there is reason to believe that some of the reported shortages were exaggerated in order to insure maximum public compliance with the restrictions.

There was no material improvement in the situation through the fall of 1953, and by January 1954 there were 41 communities with serious water problems in southern Illinois.

Table 1 shows by months, beginning with January 1954, how many communities were faced with shortages of supply from deficient surface-water sources or from seriously diminished yields caused by low water levels in wells. The data given summarize conditions at the end of each month. The standards for determining whether a community was faced with shortage were: (1) restrictions on use enforced by local authorities as a result of apparent inadequacies; or (2) in the case of surface-water sources, less than six months' supply on hand.

Although the situation improved during the winter of 1953-54 there was a gradual increase in the number of communities experiencing shortages through the summer of 1954. During the fall, water requirements dropped. At the same time rains in September, October, and December helped several communities through critical periods.

During the period 1953-55, 8 communities hauled water, 8 developed supplemental groundwater supplies, and 13 laid pipelines to emergency sources. Seven communities built additional reservoirs, but had to wait many months before the new reservoirs impounded water sufficient for use.

TABLE 1

NUMBER OF COMMUNITIES FACED WITH WATER SHORTAGES, AND SUPPLEMENTARY SOURCES IN USE, BY MONTHS

	Number of Communities			Supplementary Sources In Service	
	Surface	Wells	Total	Temporary	Permanent
January 1954	31	10	41	3	--
February	26	8	34	3	--
March	--	--	--	--	--
April	22	6	28	12	--
May	22	6	38	8	5
June	25	6	31	12	3
July	26	8	34	14	4
August	27	13	40	14	1
September	26	8	34	11	5
October	19	8	27	5	2
November	11	8	19	10	3
December	17	6	23	5	--
January 1955	10	5	15	6	4
February	11	7	18	4	3
March	0	6	6	2	0

-- indicates data not available

While the maximum number of communities facing shortages during any single month in 1954 was 41, there were 75 communities that faced shortages at one time or another during the period 1952-55.

Table 1 also gives information on supplementary sources placed in service to relieve shortages. These sources were especially developed for use during the drought to supplement existing sources. Some of the temporary sources were subsequently converted into permanent installations for future use. Others were either exhausted, or abandoned for other reasons. From March to October 1955 there continued to be shortages at four communities, but these appear to be due to source inadequacies that were apparent prior to 1952.

Collection of Data

Rainfall deficiency mapping was commenced in the fall of 1952, and carried on throughout 1953 and 1954. This mapping program, plus reports from normal Water Survey field work, newspaper clippings, and conferences with representatives of other State agencies and persons from various sections of the State, enabled maintenance of a continuous picture of the extent and severity of the drought.

In the fall of 1953 studies were begun of frequency of rainfall deficiencies at Mt. Vernon, Olney, Urbana, Springfield and Quincy. These studies were continued throughout 1954 and into 1955. Studies of low-flow duration and frequency were commenced in the summer of 1954.

In November 1953, a tabulation of surface water supplies in Illinois (excluding those using Lake Michigan as a source) was made. It showed that there were 102 communities using surface-water sources. Of these, 17 drew water from major rivers such as the Ohio and Mississippi, whose flow was sufficient to preclude shortages. The remaining 85 communities were visited, and data were collected on adequacy of source, nature of source, water levels, pumpage, etc. Reports summarizing results of this field survey were submitted through state administrative channels in December 1953 and January 1954.

In January 1954 a program of regular monthly data collection through visits, correspondence, and telephone communications with communities affected by the drought was set up. A report of the status of the drought was prepared monthly. Each report included reviews of: precipitation in the drought region during the preceding month; streamflow and groundwater conditions; and the status of municipal water supplies. While a continuing check has been made, the program of reporting each month was terminated July 31, 1955.

Scope of Report

During 1954 it was observed that difficulty with water shortages appeared to occur most frequently with impounding reservoir supplies. Special effort was therefore made to assemble information that would be helpful toward the prevention of such difficulties in the future. The present report is therefore largely devoted to hydrologic information directly applicable to design and appraisal of impounding reservoir sources of supply. Some of the information is relevant to design of side-channel reservoirs, and some sheds light on the reliability of sources of supply like those using direct intakes with no storage.

The report deals with a small part of the total information available. Although the data used appear to have sufficient homogeneity to indicate certain general conclusions with respect to impounding reservoir supplies, the findings of this report may not be considered as a substitute for a detailed examination of all the local records for design of a specific project, since local conditions can cause considerable deviation from a regional average.

Acknowledgments

This study was carried out during the administration of A. M. Buswell, Chief of the State Water Survey Division. The writers are indebted to H. F. Smith, B. O. Larson and J. B. Stall of the Engineering Subdivision for critical comments and helpful suggestions throughout the course of the study. Robert and Ronald Russell did the drafting for this report and Dhiru N. Desai and Ray Schuster aided greatly in tabulation and computation work.

The writers are indebted to J. H. Morgan, District Engineer for Illinois, Water Resources Division, U. S. Geological Survey, and to his operational assistant Warren Daniels, Hydraulic Engineer, for making available provisional streamflow data used in this study much earlier than such data are ordinarily disseminated.

The authors are grateful for the review of a preliminary draft and for the valuable suggestions given by: W. D. Mitchell, J. H. Morgan, and Richard Godfrey of the U. S. Geological Survey; Professors J. J. Doland and V. T. Chow of the University of Illinois; to Paul Sutton of the U. S. Weather Bureau; to R. D. Wilson, Consulting Engineer; and to Floyd Huff, N. L. Rupani, H. F. Smith, J. B. Stall, Glenn E. Stout, and Max Suter of the State Water Survey staff.

Throughout this study, there was excellent cooperation between representatives of the Division of Sanitary Engineering, Illinois State Department of Public Health, and members of the State Water Survey staff. By means of working arrangements between these two agencies, information gathered

on municipal situations throughout the drought region was exchanged freely through letters, reports, and conferences. The information furnished by Health Department engineers was a substantial factor in making it possible to keep the necessary field work within the limits of the personnel available for the study.

In the course of the regular visits to the cities that had water supply difficulties during the period 1952 to 1955, information was obtained from the waterworks operators, water superintendents, and city officials. In many instances the visits stimulated local officials to take regular observations. These observations were most helpful in the study. The local officials, who were very cooperative about assembling information on pumpage, water use, reservoir levels, etc., are too numerous to name, but their assistance was invaluable. Similar fine cooperation was extended by consulting engineering firms, which were extremely helpful in providing current and past data on communities that had retained them.

HYDROLOGIC ASPECTS OF WATER SUPPLY

Evaluation of the many problems encountered in public and private water supplies during the drought of 1952-55 requires knowledge of the water resource conditions that ordinarily prevail in Illinois. These conditions include the types of sources developed for water supply, the usual rainfall and runoff patterns in the State, the normal seasonal variations in availability of moisture, and particularly, the usual practice in design of impounding reservoir supplies. With a few exceptions, existing installations in Illinois functioned satisfactorily when conditions were normal, or when more water was available than usual. This was the condition from 1944 to 1952. The paragraphs under this heading describe average conditions in the State.

Sources in Use

There is much variation in the nature of the water resources throughout Illinois. Figure 1 shows a generalized map of the water resource provinces of the State. This map indicates that groundwater may be obtained readily from bedrock wells in the northern quarter of the State, but that bedrock wells are much less extensively available elsewhere. Most of northern Illinois is covered by thick glacial deposits[2] in which sand-and-gravel-wells are frequently constructed. Some of these wells have capacities exceeding 1000 gallons per minute. These glacial deposits readily receive and yield water. They therefore play a role in the reception of precipitation, and in its discharge to streams in those portions of the State where these deposits are thick and permeable.

The glacial deposits are thinner and less permeable in the southern part of Illinois, and there the storage and travel of water underground are generally very limited. As a result, groundwater contributions to streamflow are generally smaller in the southern part of the State than in northern Illinois.

Extensive alluvial deposits, associated with many of the major streams of the State shown in Figure 2 increase groundwater discharge and availability. In addition to these alluvial deposits there are buried valleys containing sand-and-gravel formations at many locations in the State[3], which also increase local streamflow and groundwater availability.

Figure 3 shows the location of municipal groundwater supplies in Illinois as of 1948. It will be noted from this figure that there is a heavy concentration of groundwater sources in the northern portion of the State, and that these sources are less common in southern Illinois. Figure 4 shows the public surface-water supplies in Illinois. These are seen to be much more common in the southern portion of the State than in the north, where groundwater is generally available in sufficient quantity for municipal purposes.

Comparison of Figures 2 and 4 shows that many communities are located near major rivers in the State. Some of them can draw water from these rivers without raw water storage facilities. Small storage facilities, in the form of channel dams, are common along some of the larger secondary streams. There are few streams large enough to yield water supplies to direct intakes or channel dams in southern Illinois. Impounding reservoir sources and side-channel storage reservoirs, as shown by Figure 4, are therefore numerous.

A side-channel storage reservoir is a reservoir having a relatively small tributary drainage area, into which water is pumped from a large stream during those periods when the flow in the stream is sufficient. The rates of pumping into side-channel reservoirs vary from amounts slightly in excess of the average daily community demand to rates of pumpage 20 times the demand. Many municipal supplies in the southern half of Illinois began with impounding reservoir sources, which relied upon the runoff from the watershed upstream of the reservoir. As Figure 4 shows, there is still a large number of these, but many of them have been converted into side-channel storage units by the installation of pumping facilities on larger streams nearby. Some of them are in an intermediate state of development. Thus, it is difficult to decide whether to classify them as side-channel or as impounding reservoir sources.

Table 2 shows the sources of supply of public water systems in the 66 counties in the drought zone. These counties are located south of line AA_1 in Figure 4. Line AA_1, was located after study

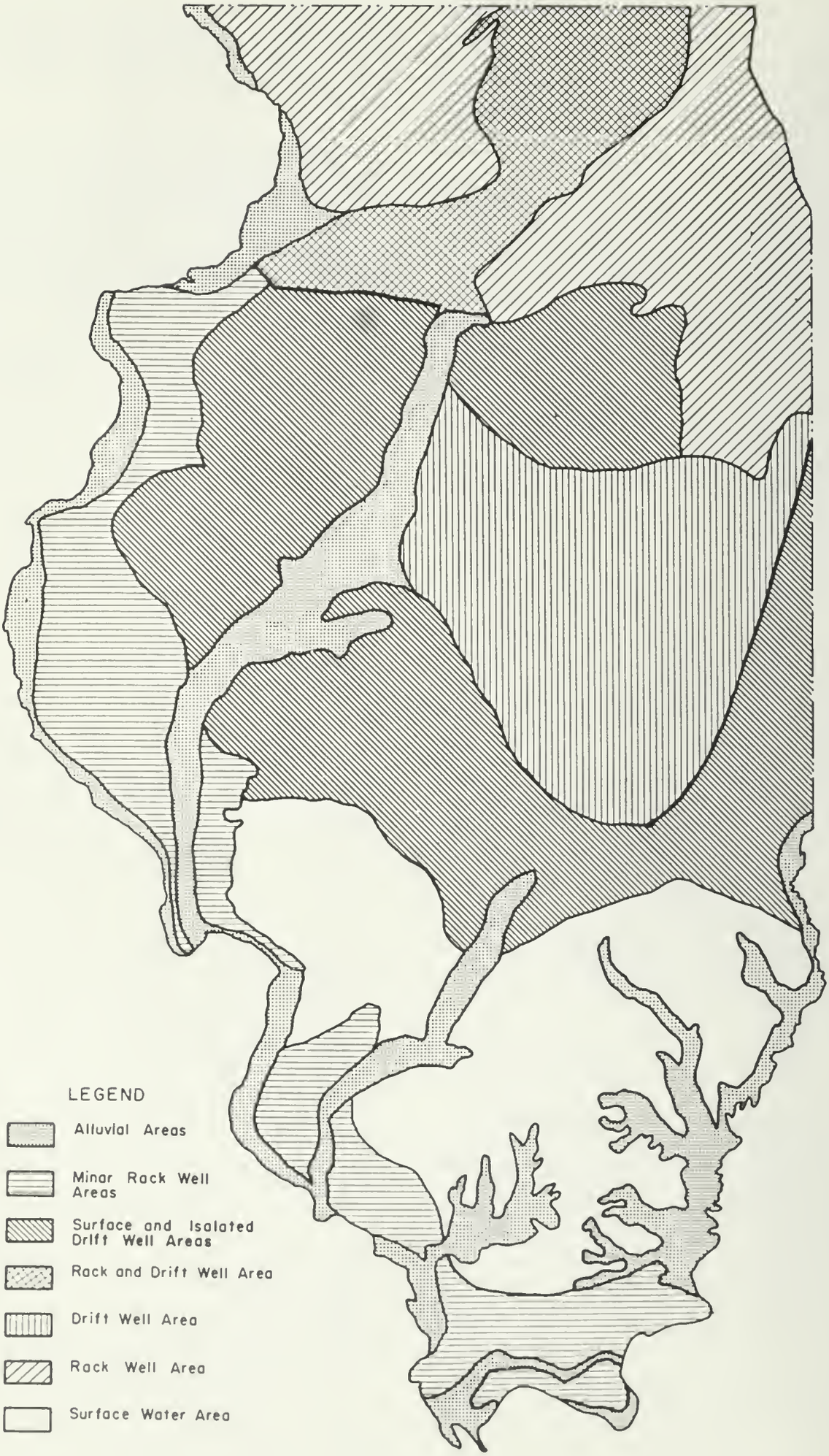


FIGURE 1. GENERALIZED MAP OF WATER SOURCE PROVINCES IN ILLINOIS.

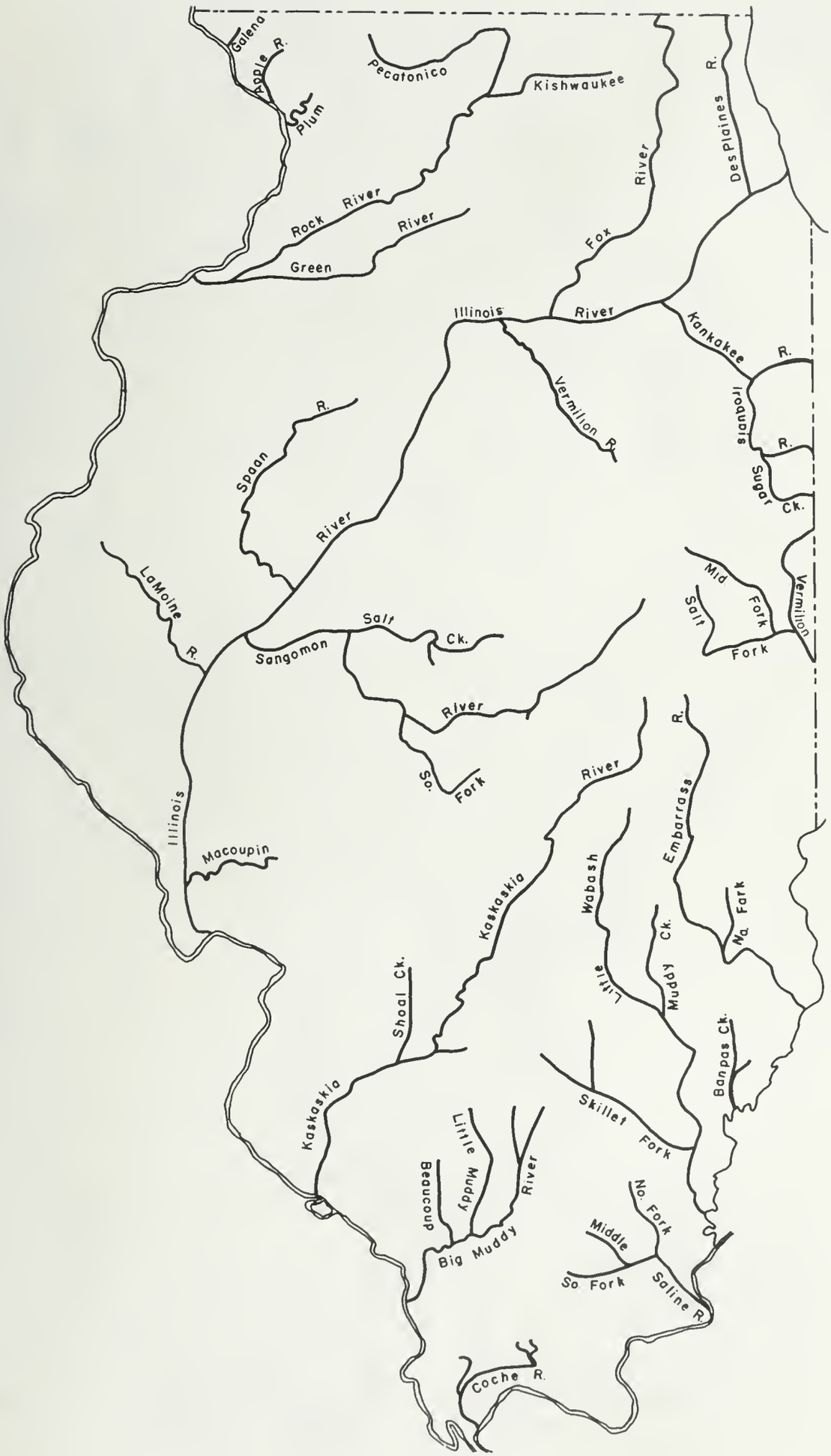


FIGURE 2. MAJOR STREAMS IN ILLINOIS.

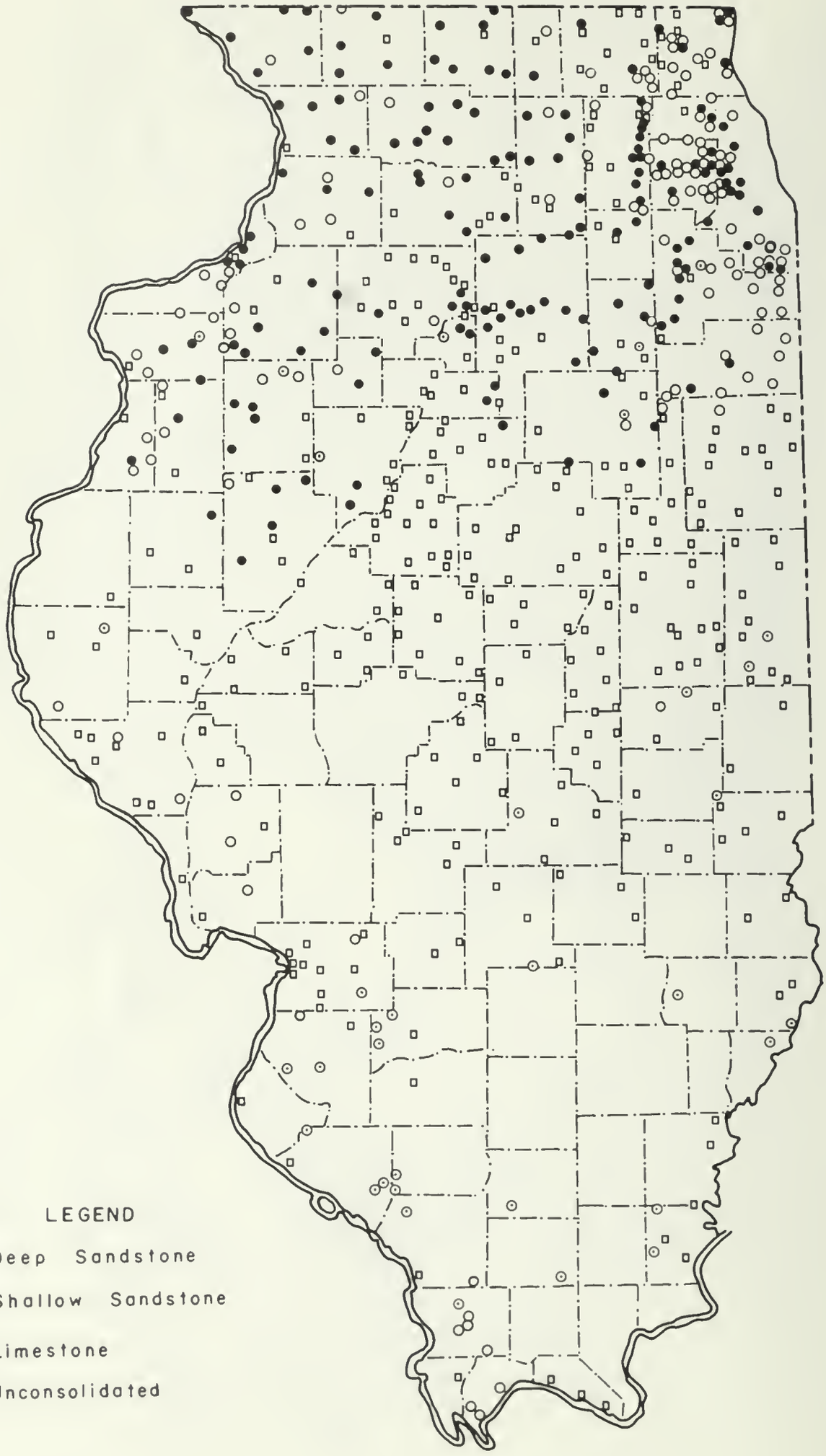


FIGURE 3. SOURCES OF PUBLIC GROUNDWATER SUPPLY IN ILLINOIS.

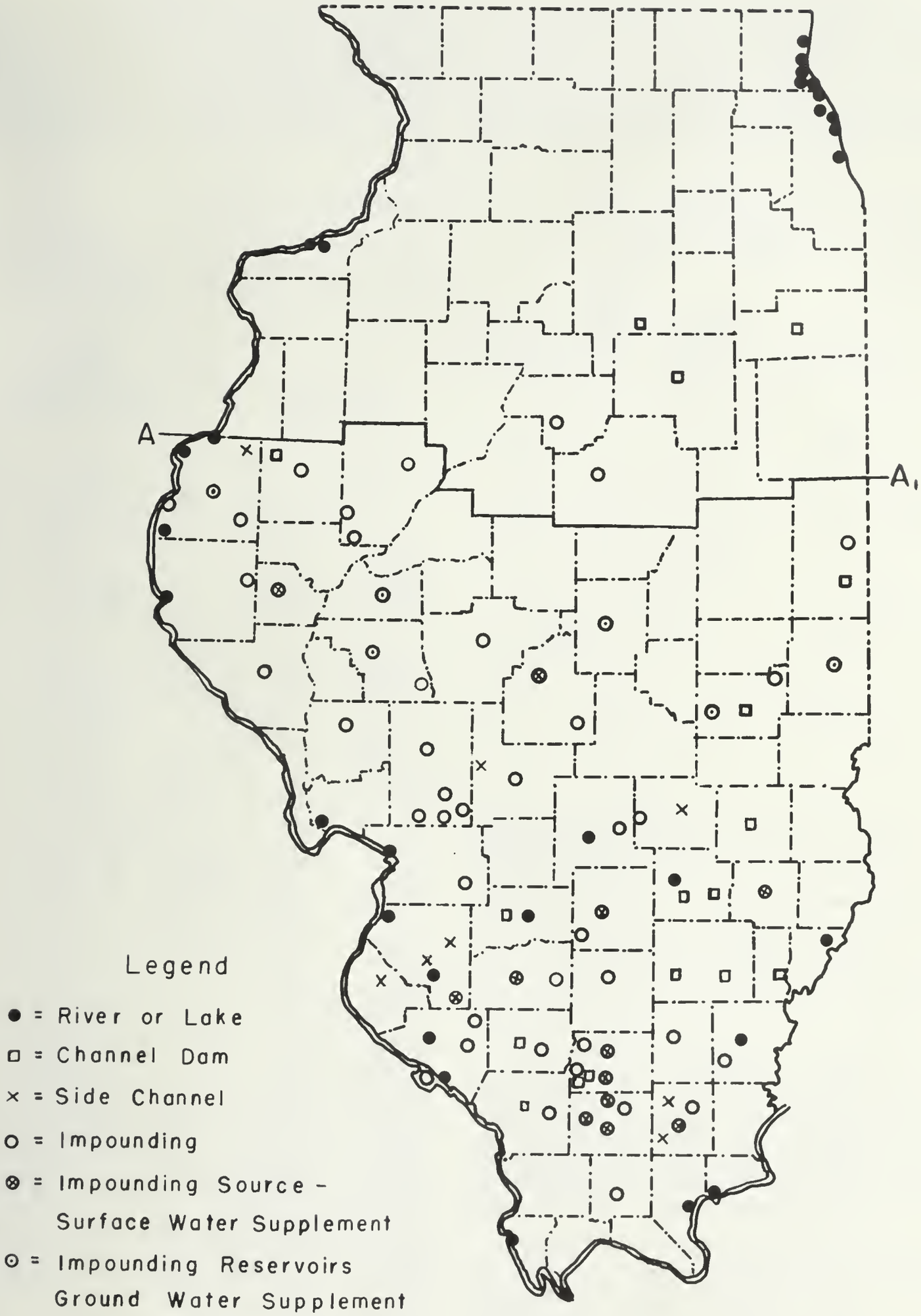


FIGURE 4. SOURCES OF PUBLIC SURFACE WATER SUPPLIES IN ILLINOIS.

of runoff and rainfall data revealed that the effects of the 1952-55 drought were felt throughout the area below the line.

Also listed in Table 2 are the numbers of communities in each source category that had water shortage difficulties during 1952-55. Since the data indicate an extremely high incidence of water source difficulties with impounding reservoirs, the information available on impounding reservoirs with which difficulties were experienced has been assembled and is presented in Table 3. This gives the year of construction for each of the major reservoirs in use in 1952, watershed area, surface area, capacity, average depth, and inches of runoff required to fill the reservoirs at the time of their construction.

TABLE 2

SOURCES OF SUPPLY FOR PUBLIC WATER SYSTEMS IN THE DROUGHT REGION

Source of Supply	Number of installations	Number of communities having shortages 1952-55	Per Cent having Shortages
Groundwater			
Deep sandstone	12	0	0
Shallow sandstone	16	2	12
Limestone	14	4	29
Unconsolidated	160	16	10
Total	202	22	11
Surface Water			
Direct Intake	18	0	0
Channel Dam	15	7	46
Side Channel Reservoir	7	6	86
Impounded Reservoir	58	40	69
Total	98	53	54

An analysis of these data in Table 3 indicates that the median watershed area was 2.5 square miles, and that 90 per cent of the communities use drainage areas of less than 18 square miles. The median runoff required to fill these reservoirs was 3.3 inches. Extreme low and high values of runoff required to fill reservoirs were, respectively 0.34 inches and 14.7 inches. Median of the average depths was 8.65 feet.

Normal Rainfall in Illinois

The normal precipitation pattern, from the 45-year averages for the period 1898-1942 provided by the U. S. Weather Bureau [4] is shown in Figure 5. From this illustration it will be seen that rainfall in the State varies from less than 34 inches per year in a portion of northern Illinois to more than 46 inches per year in part of southern Illinois, a geographic variation of more than 35 per cent. In many years, as in the period 1952-55, the rainfall does not approximate the normal pattern.

Normal Runoff in Illinois

Several generalized maps for the United States indicate the distribution of runoff in Illinois [5] [6], but no detailed mapping of Illinois mean annual

TABLE 3

IMPOUNDING RESERVOIR SOURCES FOR COMMUNITIES FACED WITH WATER SHORTAGES 1952-55

City	Year Built	Watershed Area Square Miles	Lake Surface Area, Acres	Original Capacity, Million Gallons	Average Depth, Feet	Runoff Required to fill Reservoir Inches
Altamont	1935	2.9	30	75	7.7	1.5
Ashley	1941	1.24	21	49	7.1	7.4
Astoria	1924	0.47	6.07	28	14.1	3.4
Augusta	1945	3.40	14	35	7.6	0.59
Benton	1937	2.6	100	284	8.7	6.3
Bunker Hill	1937	7.19	27.6	43	4.8	0.34
Carterville	1923	2.3	45.0	154	10.5	3.8
Carthage	1926	2.9	39.7	133	10.2	2.6
Centralia	1910	6.87	261	1064	12.5	8.8
	1942	38.0	960	1920	6.1	2.9
Christopher	1925	0.6	20.0	60	9.2	5.7
		1.50	40.0	160	12.2	5.6
Coulterville	1944	1.22	26.8	65	7.4	3.0
Decatur	1922	906	2747.0	6340	7.1	0.4
DuQuoin	1937	10.0	250.4	900	11.0	5.2
Eldorado	1920	2.23	98.0	258	8.1	6.6
Elkville	1937	1.5	62	175	8.6	6.7
Gillespie	1923	5.73	70.5	260	11.3	2.6
Hillsboro	1918	7.5	96.0	500	15.6	3.8
Jacksonville	1939	24	250.0	590	7.2	1.4
Johnston City	1922	3.85	65	149	7.0	2.2
Kincaid	1940	2.4	36	89	7.6	2.1
Marion	1921	6.48	110	231	6.4	2.4
Marissa	1938	0.6	--	25	--	2.4
Mattoon	1937	18	240	1000	12.7	3.2
McLeansboro	1912	1.52	35	228	19.9	8.6
Mt. Sterling	1935	1.8	35	100	8.7	3.2
Mt. Vernon	1900	9.2	128	760	18.2	4.7
	1944	4.65	147	521	10.9	6.4
Nashville	1936	1.39	39.5	104	8.1	4.3
Norris City	1937	0.83	21	46	6.7	3.2
Olney	1903	0.78	34	200	18.0	14.7
Paris	1937	17.7	180	315	5.4	1.0
Pittsfield	1923	1.84	46.0	120	8.0	3.7
Pana	1949	8.4	218	1006	14.1	6.9
Sesser	1925	1.1	45.0	54	3.7	2.8
Sparta	1915	1.2	35.0	105	9.2	5.0
St. Elmo	1935	3.0	14.3	36	7.7	0.6
Vienna	1937	2.34	21.0	35	5.1	0.86
Virginia	1933	1.5	19.0	60	9.7	2.3
West Frankfort	1946	3.8	149	354	7.3	5.3
White Hall	1897	0.97	34	150	13.5	8.9

runoff has been done. For the purposes of this report, an approximation of runoff in Illinois was obtained by using the average data given in the study by Mitchell [7]. Mitchell's data are for 25 stations having periods of record ranging from 9 years to 35 years, but most of the data are for periods of 15 years or more.

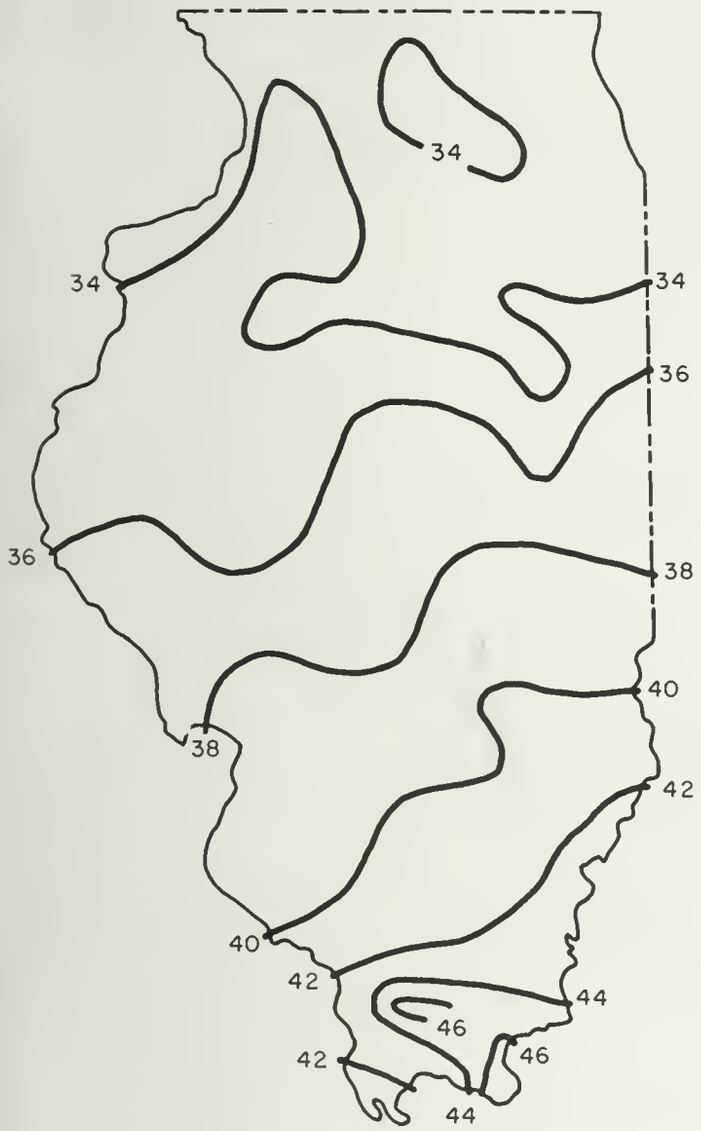


FIGURE 5. 45-YEAR AVERAGE ANNUAL PRECIPITATION IN INCHES.

Drainage basins for these stations reach into Wisconsin in two instances and into Indiana in one. Using these data, a map of mean annual runoff was prepared for the State (Figure 6). This map, which generally agrees with previously published information, indicates a considerable geographic variation of runoff within the State, ranging from less than 8 inches in the west and northeast portions of the State to more than 16 inches in the Shawnee Hills.

The unit usually used for quantitative expression of streamflow is cubic feet per second. As a rule this unit is applied to the entire flow at the gaging station, but it is sometimes converted to cubic feet per second per square mile of drainage area. This is equivalent to a unit of length per unit time. It is therefore proper to express runoff (or streamflow, or discharge, which are synonyms) in suitable length units such as inches per unit

FIGURE 6. MEAN ANNUAL RUNOFF IN INCHES PER YEAR. 20 to 40 Year Records for 25 Stations.

time. Such a unit has the advantage of being directly comparable with rainfall measurements, which are customarily expressed in inches, and are therefore widely recognized. Accordingly, throughout this report, streamflow has been expressed in inches occurring during stated periods of time.

It would be desirable to prepare such a map as that shown in Figure 6 for a uniform period of time and with more detailed information. This could be done either by choosing a representative recent short period for which abundant records are available, or by using a longer period and

extending[7] the discharge values for the stations of short record. Inasmuch as there are 165 gaging stations in the State, the preparation of a detailed map was impractical for the present report.

Seasonal Variances in Hydrologic Cycle

Water that reaches the earth's surface as precipitation continues to be in motion. Its destination is determined by a number of processes, which include infiltration, evaporation, transpiration, underground travel and detention, percolation, and overland flow. The effect of each process on the others varies seasonally. These effects may best be studied by beginning with data available for measured processes (rainfall and runoff), and then calculating the effects of the less-well-determined processes (evaporation and transpiration) in order to evaluate the large unknown - underground storage.

Monthly averages of runoff from five stream-gaging stations in the drought region were prepared, together with monthly averages of rainfall as indicated by rain-gaging stations in or close to their drainage areas. The five monthly averages were consolidated into an arithmetic mean for each month for rainfall and similarly for runoff. The stations used are listed in Table 4. Average annual discharge values are for the period of record listed.

TABLE 4

STATIONS USED IN STUDY OF SEASONAL VARIATIONS IN HYDROLOGIC CYCLE

Stream-Gaging Station	Watershed Area in Square Miles	Years of Record to 1955	Average Discharge, Inches per Year	Rain Gaging Station
Big Muddy at Plumfield	753	41	13.16	Mt. Vernon
LaMoine at Ripley	1310	34	7.79	Quincy
Little Wabash at Wilcox	1130	41	10.17	Effingham
Macoupin Creek at Kane	875	35*	8.76	Carlinville
Sangamon at Monticello	550	45	10.28	Urbana

*7 years estimated

The monthly data are shown graphically in Figure 7. As the illustration shows, some of the seasonal variances in rainfall are reflected in the runoff. The low precipitation in February seems to coincide with a reduction in runoff during that month. However, runoff does not increase proportionately to rainfall in March since infiltration begins with the thawing of the soil, usually in that month. A similar disproportion is noted in April when vegetative demands commence and temperature rises have induced a substantial increase in evaporation of soil moisture. The disparity between rainfall and runoff curves therefore becomes greater until cooler weather in the fall brings a decline in transpiration and land evaporation rates.

The difference between the rainfall and runoff curves, which is shown in Figure 7, represents the amount of water available each month for transpiration and evaporation. A second curve has

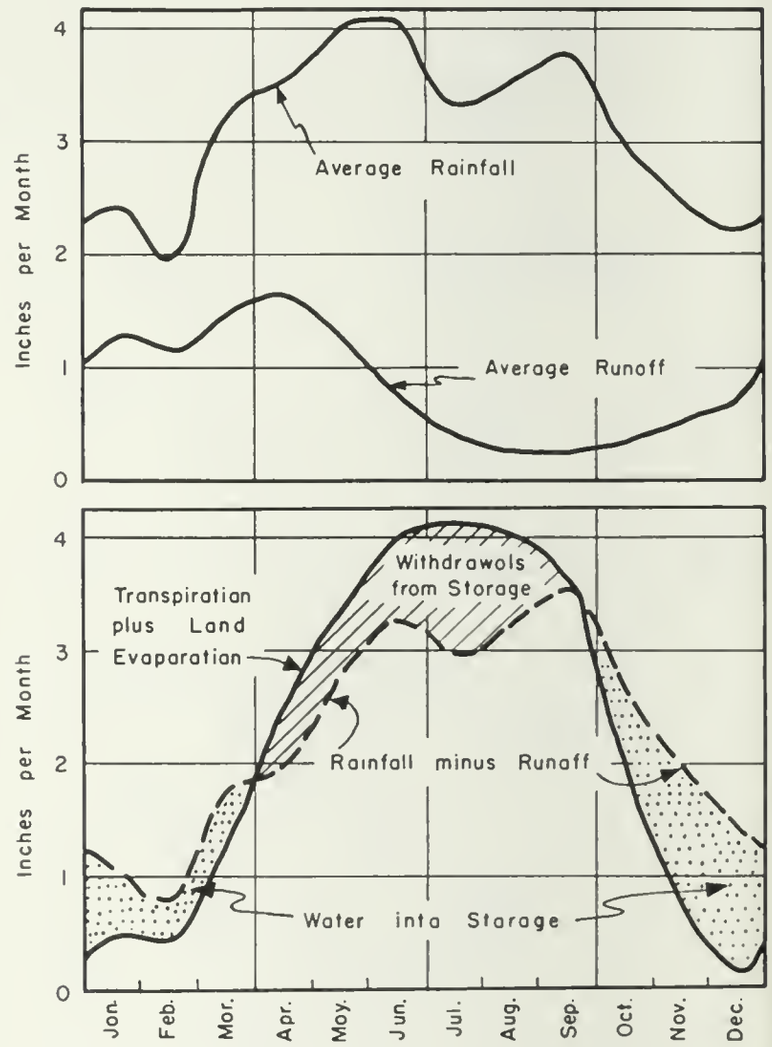


FIGURE 7. SEASONAL VARIATIONS IN THE HYDROLOGIC CYCLE. Southern Illinois.

been computed which shows the calculated evaporation plus transpiration[8], based on rainfall and temperature data. The areas between the curves in Figure 7 represent water taken from storage in the earth or returned to storage. From the areas between these curves, it may be seen that from the first of April until mid-September, the vegetative and evaporative demands exceed the water available. This difference must be made up either by water from the soil or by reduced transpiration and evaporation. It is noted that the greatest rate of withdrawal from storage occurs in July.

From mid-September until the end of March, in the average year, water returns to storage. From

the behavior of groundwater hydrographs for shallow wells, which commonly show no particular recovery until the end of November or later, it is judged that the excess available water for September, October, and November largely goes to make up deficiencies in soil moisture above the water table and does not add to storage below the water table, which recovers in the period between December 1 and April 1. Considerable variations from these general relationships will be found, depending on local infiltration rates, underground moisture storage capabilities, etc.

From these curves it may be seen that in the region studied, vegetative and evaporative demands are so great that excessive rainfall is required through July and August to prevent recession of groundwater levels and reduction of soil moisture storage. In abnormally dry years, when great deficiencies in soil moisture occur, recoveries of groundwater levels and of streamflow may be delayed by several months. Such a situation had occurred in the drought region at the close of 1953 and continued to intensify until the fall of 1954. Unfortunately, the soil moisture storage capacity in the drought region is lower than that in the northern half of the State where it may exceed 10 inches[9].

CAUSES OF WATER SHORTAGE

A number of factors other than the occurrence of an unusual drought have contributed to the water shortages occurring during 1952-55. Among those which contributed directly are; population increases, increases in domestic and industrial water requirements, reservoir siltation, and declines in capacity of wells. Factors that contributed indirectly to the occurrence of shortages were municipal finance problems, deficiencies in hydrologic data, lack of advance planning, and lack of records of waterworks operation. Population growth and increases in industrial and residential water use are closely linked. There is a relationship between sediment production and runoff into reservoirs but, in years past, hydrologic data have been insufficient to determine this relationship. Municipal finance problems and insufficiency of operating records seem to be separate topics not closely related to the other factors mentioned above.

Onset of Shortages

There are distinctive differences between the behavior of groundwater and surface-water sources of supply with respect to their adequacy. A shallow, locally-recharged groundwater-producing formation that is being used to capacity is usually incapable of furnishing water at rates greatly in excess of its perennial capacity except for very short periods. Peak loads on such sources are therefore apt to produce occasional low pressures in distribution systems even though the source is adequate for the average use of the community.

Groundwater sources of supply of too-limited capacity demonstrate their inadequacy by a continuing decline of water well levels. Over-use of water from these groundwater-producing formations is therefore quite evident to the operator (if he makes regular measurements of water levels), and frequently becomes apparent to the consumers on peak days through occasional shortages of supply even in years of normal precipitation. In years of deficient precipitation, declining water levels and increasing frequency of unsatisfactory service on days of peak demand warn the community of the inadequacy of the source.

On the other hand, surface water sources that rely upon streamflow are designed to meet dry weather conditions, when streamflow may be only a fraction of the normal flow. Periods of very low streamflow occur comparatively seldom and are interspersed with periods of normal or above-normal streamflow, during which the source of supply is capable of yielding as much as two to ten times more water than may safely be taken during a critical period of low flow. Accordingly, except for deficiencies in pumping and distribution facilities, a surface water source of supply may give little indication of inadequacy even in years that are considerably below normal. It is possible during normal or near-normal years for pumpage from an impounding reservoir supply to be increased several-fold above the long-term safe yield of the supply without causing signs of shortage. Accordingly, during periods such as the relatively wet period from 1944 to 1952, symptoms of inadequacy of surface water sources may not occur unless the demand has very grossly outgrown the safe yield. (The relative wetness of the 1944-52 period became apparent during the course of this study, through examination of data such as those shown in Table 9).

Increases in Water Demand

General. Total groundwater pumpage for municipal supplies in Illinois increased from 40 million gallons daily in 1900 to about 106 mgd in 1940[10]. From 1940 to 1948 pumpage increased to 135 mgd, or about 28 per cent. The upturn in total pumpage after 1940 was very sharp. A similar striking increase of more than 40 per cent for the period 1940-1947 was reflected in the average for the five communities: Aurora, Champaign-Urbana, Dekalb, Elgin, and Rockford. Most of this increase took place following the World War II, when it occurred almost explosively.

Indications of rapid growth in demand caused Decatur to revise the plans prepared in 1948 for expansion of its water system. A survey of industrial and other uses in Decatur completed in 1954, indicated that the average daily pumpage had increased nearly 40 per cent from 1947 to 1953, and was expected to increase from 10 mgd in 1953 to

13 mgd by 1955, an increase of approximately 30 per cent. At Carthage, Illinois, pumpage increased from 0.130 mgd in 1945 to 0.225 mgd in 1950, more than 70 per cent.

A number of factors contribute to the increases in water demand that have occurred in recent years. Most obvious of these is the population growth, but increases in water demand have generally been greater than could be explained simply by greater population. A preliminary review of available data from Illinois communities in the region affected by the drought indicates that an increasing proportion of the urban population is receiving service from public water supply systems. Whereas in some communities, as few as one-quarter of the residences were connected to the public water supply system in 1930, this proportion has been rising and approaches 100 per cent in some of the communities. Industrial requirements for water have also increased as a result of the development of technological processes requiring water for cooling and processing purposes. Increased plumbing facilities and water-using devices in the home have also caused a growing demand. These devices include automatic washing machines, garbage grinding units, and air conditioners. Factors affecting water use are being given detailed study by Water Survey engineers and will be discussed in a subsequent report.

TABLE 5

POPULATION AND PUMPAGE CHANGES AT SELECTED MUNICIPALITIES IN THE DROUGHT REGION

City	Population 1950	Population Per Cent Change 1930-1950	Pumpage 1953 (1000 gpd.)	Pumpage Per Cent Change 1938-1953
Altamont	1,580	+ 29.8	75	+ 87
*Arcola	1,700	+ 0.8	90	+ 50
Ashley	738	- 4.4	75	+ 370
*Assumption	1,466	- 5.6	85	+ 41
Astoria	1,308	+ 10.0	70	+ 133
*Barry	1,529	+ 1.5	76	+ 110
Blandinsville	918	- 3.6	34	+ 162
Bunker Hill	1,238	+ 30.7	60	---
*Camp Point	969	- 3.1	23	+ 130
Carrier Mills	2,252	+ 5.2	75	+ 243
Carthage	3,214	+ 43.4	240	+ 200
Decatur	66,269	+ 15.3	10,000	+ 43
*Dongola	704	+ 10.8	48	+ 435
Effingham	6,892	+ 38.4	600	---
Elkville	934	- 17.5	45	+ 125
Eureka	2,367	+ 54.3	175	+ 75
Galatia	933	0	35	+ 400
*Greenfield	987	- 4.9	45	+ 137
Harrisburg	10,999	- 5.3	700	+ 75
Highland	4,283	+ 29.0	400	+ 92
Jacksonville	20,387	+ 14.8	1,900	+ 90
Johnston City	4,479	- 24.7	170	0
*Jonesboro	1,607	+ 29.3	70	+ 133
Kincaid - Bulpitt	2,169	+ 6.4	210	+ 600
LaHarpe	1,295	+ 10.2	70	+ 133
McLeansboro	3,008	+ 39.1	280	+ 180
Marissa - Tilden	2,558	+ 57.0	85	+ 430
Mattoon	17,547	+ 19.9	2,325	+ 86
Mt. Sterling	2,246	+ 30.2	150	+ 178
Nashville	2,432	+ 8.4	300	+ 900
Norris City	1,370	+ 23.5	80	+ 209
Olney	8,612	+ 40.2	700	+ 95
Pittsfield	3,564	+ 51.2	275	+ 244
Sesser - Valier	2,894	+ 38.7	200	+ 100
Sparta	3,576	+ 5.6	250	0
*Teutopolis	919	+ 28.7	35	+ 85
Vienna	1,085	+ 24.1	46	+ 229
Virginia	1,572	+ 5.2	100	+ 300
Waterloo	2,821	+ 25.9	180	+ 445
White Hall	3,082	+ 5.2	160	+ 19
*Windsor	1,008	+ 8.7	75	+1,150

*Indicates groundwater source. All other surface sources.

Water Demand in Drought Area. Table 5 contains preliminary information on water use in the 41 communities that faced possible shortages in January 1954. It is to be noted that the population changes for the period 1930 to 1950 were considerable in some instances, but none of the population increases exceeded 57 per cent, and in 24 instances the increases were less than 20 per cent. Pumpage data for these communities were available from field visits in 1953-54 and from other sources for 1938. The per cent change in pumpage has been computed and is given in Table 4 for each community. Pumpage increased 600 per cent or more in three cases and increased by more than 100 per cent in 24 cases.

Detailed study of the factors influencing the water use in these communities is being made, and no generalizations have yet been reached concerning communities that did not show large increases. It is noted, however, that a number of the communities for which pumpage increases were either small or negative, experienced population declines during the period of study. In other instances, the lack of pumpage increase was known to be linked to the inadequacy of the water source development, so that it was recognized in these communities that frugal use of water was essential to prevent a general shortage.

In some instances part of the increase can be attributed to leakage and wasteful use, but this is believed to be a relatively small proportion. Most of the increase appears to be due to two factors: (1) increase in number of service connections, and (2) increases in per capita and industrial use. In many of the communities studied, these increases have not yet fully materialized, and may be expected to develop in future years.

National Trends. The American Water Works Association surveys of 1945 and 1950 water pumpage indicate that a 10 per cent increase took place in per-capita consumption between these years. However, many communities throughout the nation, such as Oklahoma City, have noted such great increases in water use since 1940 as to necessitate substantial, ahead-of-schedule expansion of facilities[11]. A study by the U. S. Department of Commerce indicated that the national total pumpage through municipal systems would rise about 35 per cent in the decade 1950-1960[12].

Deficiencies in Hydrologic Data

Basic items of hydrologic data for water works design include information on streamflow, groundwater levels and precipitation data. The precipitation data are used at times, in combination with other information, to aid in the estimation of streamflow data when streamflow records are lacking. Precipitation data may also be of value for correlation with variances in groundwater levels.

Precipitation data have been available for a number of years from several hundred rain gaging stations in Illinois, and some of the records reach back well into the 19th century. On the other hand, stream-gaging records for use in design of impounding reservoirs are available only since about 1908 in Illinois, and few of the streamflow records reach back that far.

Groundwater data, collected on a long-term basis, are almost totally lacking in the drought region with the exception of certain measurements in areas where heavy pumpage has attracted attention to groundwater problems. Groundwater level and productivity data are not generally available throughout the southern half of Illinois covering a sufficient length of time and a sufficient number of locations to enable precise design of dependable groundwater sources of supply.

Inasmuch as data previously given have indicated that the highest proportion of difficulties in water supply during the 1952-55 drought occurred at communities having surface water storage, and since streamflow data are essential for design of storage facilities for such installations, it is clear that the most critical hydrologic data with respect to water supply development are those gathered from stream-gaging.

An analysis has been made of the 144 streamflow gaging stations operated within the State of Illinois as of January 1, 1955, classifying stations by drainage area and date of initiation of measurement program. These data are shown in Table 6 which indicates that of the 144 stations, only 30 are for drainage areas less than 20 square miles. Table 3 shows the distribution of drainage areas of impounding water supplies that experienced difficulty in 1952-54. It will be seen that 90 per cent of the drainage areas used were smaller than 18 square miles.

Table 6 shows that no stream-gaging stations now in operation on drainage areas of less than 50 square miles were established prior to 1931, and that only eight stations had been established prior to 1940 on drainage areas of less than 50 square miles, as compared with 43 subsequent to 1940.

These data reflect the growth of interest in the gaging of flows from small drainage areas since 1930.

As the population and demands for water grow in the future, it would be expected that larger drainage areas will be required than those now in use. Taking this into account, it would appear desirable, in the region where surface waters comprise the major available sources, to increase considerably the collection of streamflow data for drainage areas of 50 square miles or less.

TABLE 6

CLASSIFICATION OF STREAMFLOW MEASURING STATIONS IN OPERATION IN ILLINOIS, JANUARY 1, 1955

Year Started	Number of Stations on Drainage Basins of Various Areas, (sq. mi.)					
	0-5	5-10	10-20	20-50	50-100	100+
Before 1921	0	0	0	0	0	23
1921-30	0	0	0	0	1	7
1931-40	1	2	1	4	5	27
1941-50	2	8	8	7	3	31
1951-54	3	1	4	2	1	3
Total	6	11	13	13	10	91

Observations from stream-gaging stations are of limited value for design purposes until they have been collected for a period of years. In the past, it has been considered that 20 years is a minimum time for obtaining trustworthy streamflow records at a given location. The fruits of the increase in stream-gaging immediately prior to 1940 are therefore now becoming available but, until the last two or three years, little use had been made of these data for design.

Data from the five reservoirs on which the Illinois State Water Survey has been keeping records since prior to 1930[13] had not been widely disseminated when most of the present reservoirs were built. Drainage areas for these reservoirs range from 2.95 to 61 square miles.

Study of the dates of construction (and presumably of design) of the sources for the 53 surface-water-using communities that experienced difficulty during the 1952-55 drought shows that 9 were built in the period prior to 1920; 14 in 1920-29; 19 in 1930-39; 11 in 1940-49. Thus it is seen that only 20 per cent of these reservoirs were designed after 1940, when the collection of small-basin streamflow records was intensified.

It is therefore clear that the streamflow data necessary for design of reservoirs were not available at the time when most of the Illinois municipal water supply impoundments were constructed. Far better design information is now available but there is still question as to whether appropriate information is being collected for economical and

safe design of the impounding reservoirs that will be needed for water supply in Illinois.

Municipal Finance Problems

Problems of municipal finance have attracted increasing attention in recent years. The severe difficulties encountered in financing necessary local functions so preoccupied many communities that insufficient attention was paid to water supply problems. In other cases, communities that experienced shortages in 1952-55 had been aware in earlier years that their sources of supply were not adequate, but were unable to finance the necessary improvements. These communities had managed to eke out their water supplies by the application of local restrictions during the drier periods.

Of the 75 southern and central Illinois communities that have faced water source problems during 1954, 26 had developed new permanent sources of supply as of June 1955. Eight more had work under construction and 19 were planning to develop additional sources. Twenty-two had taken no action.

Insufficient Records

The low-water situations for both surface water and groundwater supplies have emphasized the desirability of maintenance by cities of careful records of water levels in wells and reservoirs. Many communities do not have the necessary equipment for determination of water levels. Equipment of this kind includes airlines in wells for determination of water levels, and staff gages on intake towers in impounding reservoirs. By means of such devices, water levels may readily be determined at regular intervals. Readings should be taken by local waterworks personnel at weekly intervals and entered in the permanent records of the system.

By analysis of these records two or three times per year, it is possible to anticipate shortages and to take appropriate action to forestall them.

Experience of Communities Affected

A classification was made of the sources of supply for the 75 communities that experienced or faced shortages in 1952-55. At these communities restrictions were placed on water use in nearly all cases. In a few instances, no local restrictions were applied, but the quantity of water on hand was sufficient to last less than six months. Such communities were therefore considered to be facing shortages.

Computation of the quantity of water on hand in storage reservoirs was begun in December 1953. In January and February 1954, communities were estimated to be endangered by shortages if the quantity of water on hand was less than an eight months' supply. As the more favorable season for runoff approached (See Figure 7) the quantity on hand to insure a safe supply was estimated to be a six months' supply. This figure could have been increased again as the likelihood of runoff lessened in late spring and early summer, but the six months' figure was maintained for purposes of uniformity. Designers in Illinois have commonly provided capacity for storing one or more years' supply of water, depending on local conditions. The six months' supply is considered a minimum amount to have on hand as a season of high runoff (January through May) approaches, and it is considered that a larger quantity would be desirable at the close of the season of high runoff.

Relation of Source to Incidence of Shortages. The data from the classification by sources are given in Table 2, which shows that only 11 per cent of the cities having groundwater sources faced shortage problems as compared with 54 per cent of those using surface-water sources.

There were no problems in the groundwater-using-communities having deep sandstone sources, although these are relatively rare in the drought region. Highest proportionate incidence of groundwater shortages was in the limestone sources of supply, but the largest group of groundwater shortage problems was in communities having wells drilled into sand-and-gravel formations.

The median depth of wells for the 16 communities using sand-and-gravel sources of supply was 40 feet. Depths of wells ranged from 28 to 130 feet for these communities, and three-quarters of the communities had wells 68 feet or less in depth. Seasonal effects and immediate local recharge play a very important part in the operation of wells as shallow as this.

There were no problems at the surface-water-using communities having direct intakes, since these communities are located adjacent to rivers whose minimum flows are in excess of the community's requirements. The remaining three categories (channel dams, side-channel reservoirs, and impounded reservoirs), all of which use artificially-produced storage as an essential element of the supply scheme, showed a very high incidence of shortages. Of this group, 68 per cent faced shortages, with the highest incidences of shortage problems occurring in the side-channel and impounding reservoir supplies. Occurrence of difficulties with these sources was so general as to make clear that either (1) the drought was of an extraordinary severity, and/or (2) the existing reservoirs were not adequate to meet the needs of the communities.

Source Capability versus Water Demand. A study was made of causes of shortages at the communities that have had difficulties related to source inadequacies during 1952-55. The history of each community's water system was reviewed to determine: (a) the change in pumpage in the period 1938 to 1953, (b) losses in capacity due to sedimentation in surface water reservoirs, and to declines in capacities in groundwater installations caused by aging of wells, and (c) the adequacy of the source, as of 1938, to meet the demands that could have been expected to develop by 1955.

In those groundwater using communities that experienced shortages, particular attention was paid to distinguishing between shortages caused by exhaustion of available groundwater and those caused simply by declining well capacities, as indicated by reductions in specific capacities.

The causes of shortages were placed in either of three categories:

1. Capacity insufficient to meet anticipated demands.
2. Decline in capacity.
3. Unexpectedly large increase in demand.

Table 5 shows some of the experiences with increases in demands for water in communities in the drought region. Median increase for the group of 41 communities was of the order of 100 per cent for the period 1938 to 1953. For classification of causes of difficulties a 100-per cent increase was therefore arbitrarily chosen as a value of pumpage increase which might usually have been anticipated during this period in the drought area.

Where the increase in pumpage exceeded 100 per cent for the period 1938 to 1953, the cause of shortage was usually considered to be unexpected increase in demand. There were two exceptions to this rule, however. One exception covered those instances in which designers of new systems had planned for increases in demand greater than 100 per cent. In these cases the shortages were attributed to other causes. The other type of exception dealt with communities whose water systems had been in service for several decades prior to 1938. Some of these appeared to have needed expansion in 1938 to meet demand increases less than 100 per cent that could be expected by 1953; the cause of their difficulty was deemed to have been insufficient capacity.

In many instances it was difficult to attribute the cause of shortage to any single factor. In several cases all three factors were operative. In each of these cases a decision was therefore made on a basis of the judgment of Survey engineers as to which was the determining factor.

Adequacy of surface water sources as of 1938 was judged on the basis of reasonably-anticipatable increases in demand and the draft-storage criteria presented later in this report. (Figure 40.)

In several instances, such as at Johnston City and Hillsboro, calculations indicated that the reservoir design was such that the system should have been capable of supplying the current community demands for droughts such as those covered by Figure 40. The fact that water shortages became imminent at these communities indicates that the drought of 1952-55 was more extreme at these locations than the low-flow periods involved in the preparation of Figure 40. Inasmuch as shortages impended at these communities, they were classified as having capacity insufficient to meet the drought. Neither Johnston City nor Hillsboro had to resort to emergency sources of supply, and at the worst of the drought, the quantities remaining on hand were sufficient to supply the communities on a restricted basis for two months at Johnston City and five months at Hillsboro.

Summary of Causes of Shortages. The analysis of the data to determine causes of water supply difficulties is summarized in Table 7. From this table it is seen that, considering all the water systems studied, the principal cause of failure was insufficient capacity, that the second most frequent cause was increased demand, and that declining yield was the least frequent cause of shortages.

TABLE 7

CAUSES OF WATER SUPPLY DIFFICULTIES. MUNICIPAL SUPPLIES IN THE DROUGHT REGION

	Surface Water Sources	Groundwater Sources	Total
Capacity insufficient to meet anticipated demands	23	15	38
Decline in capacity	4	5	9
Unexpectedly large increase in demand	26	2	28
Total	53	22	75

For surface-water-using communities the most frequent cause of shortage was unexpectedly large increase in demand, followed by insufficient capacity, and decline in capacity. In many cases two or more of these causes entered in: where there was question, the difficulty was attributed to the easily-measured increase in demand. For groundwater sources, insufficient capacity came first, declining capacity was second in importance and increased demand, third. This difference in the order of causes of difficulty appears to be due to

the fact that, in a number of groundwater-using communities where shortages did occur, peak day shortages before the drought had shown that the supplies had no excess capacity, and demands therefore did not increase. In the case of surface waters, as mentioned previously, there was frequently no such public awareness. Uses therefore tended to increase rapidly at surface-water source communities.

The second most frequent cause of shortage of surface supplies was insufficiency of reservoir capacity. These failures were due primarily to inadequacies in hydrologic data for safe design of reservoirs. Least frequent cause of shortage of surface-water source was decline in capacity (due to silting). This was the primary cause in only five communities, and only a contributing factor in many of the others.

EXTENT AND INTENSITY OF THE DROUGHT

The area within Illinois covered by the drought was generally within the southern half of the State, although moisture deficiency conditions extended northward into Hancock and McDonough Counties, west of the Illinois River, and north of Champaign County at times. As precipitation occurred, the area expanded, contracted, and shifted.

In the following studies, the data used for mapping and tabulating are monthly totals. These monthly totals are available from U. S. Geological Survey and Weather Bureau publications.

Rainfall Conditions

The drought area can be outlined by mapping rainfall data for various periods. Rainfall departure maps were prepared for various periods beginning April 1, 1952, which was chosen as the beginning date for the drought on a basis of published U. S. Weather Bureau Records[14]. From these records, preliminary values of average precipitation and departures from normal for Northern, Central, and Southern divisions of the State were obtained. March 1952 showed above-normal precipitation for all three divisions, but below-normal precipitation occurred in April and May in all three divisions. Slightly above-normal precipitation was recorded in June for all three, but below-normal precipitation occurred in the Northern and Central districts in July, and for August 1952, precipitation was below normal in both the Southern and Northern divisions. In the Southern division, where most of the 1952 water supply difficulties occurred, rainfall was generally below normal continually for the remainder of 1952[14]. For the rainfall analysis carried on during the drought, 39 of the 183 U. S. Weather Bureau rain gaging stations in Illinois were used. These particular stations were chosen because regular monthly reports for them were issued by the U. S. Weather Bureau within the first week of each month.

Since there are significant geographic variations in rainfall within the State, as shown in Figure 5, it seemed desirable to analyze the rainfall data on the basis of departures from normal precipitation rather than on the actual amount that occurred. Rainfall departures may be calculated with monthly data, using the actual rainfall for the month and the 45-year normal values published by the Weather Bureau[4]. A departure is the amount by which the actual amount differs from the normal. Departure values may be cumulated to produce total

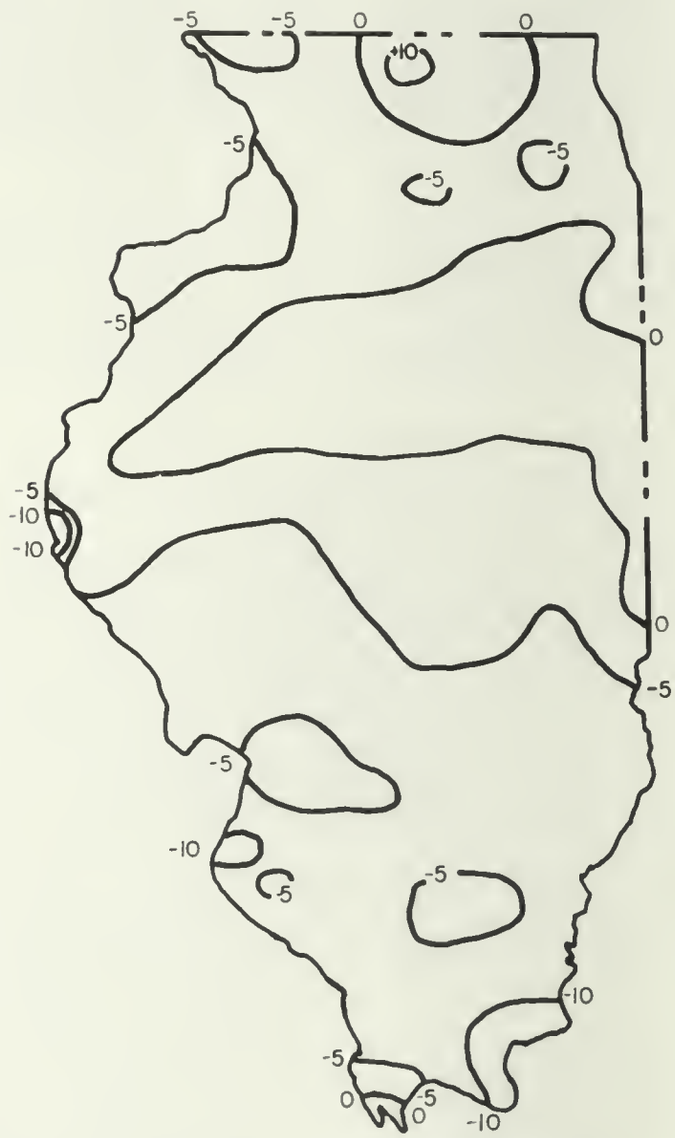


FIGURE 8. RAINFALL DEPARTURE FROM NORMAL IN INCHES. April to December 1952.

departure values for any desired period. On the maps which follow, deficiencies have been indicated by minus signs, and above-normal rainfall has been indicated by plus signs. Inasmuch as, in certain parts of the State, above-normal rainfall occurred, and since the maps show both the excesses and the below-normal rainfall, they have been called departure maps, although this report is primarily concerned with deficiency data.

As the drought extended into 1954, it was recognized that the effects of rainfall deficiency on

water supplies would not extend in full force much more than one year. This lack of full cumulative effect is due to the limiting value of soil moisture storage, which is of the order of 10 inches[9]. As deficiencies increase toward a value of 10 inches, water may be withdrawn from soil moisture storage. When the soil moisture deficiency exceeds approximately 10 inches, transpiration virtually stops and larger values of deficiency become relatively less meaningful. There are some seasonal opportunities for replenishment of the soil moisture but, with respect to soil moisture and overland flow, there is little hydrologic reason for cumulating rainfall deficiencies more than one year when they become much greater than 10 inches. Accordingly for mapping 1953 conditions, the beginning month for tabulation was shifted to January 1, 1953 and, for 1954, the deficiency computations were shifted to begin with January 1954.

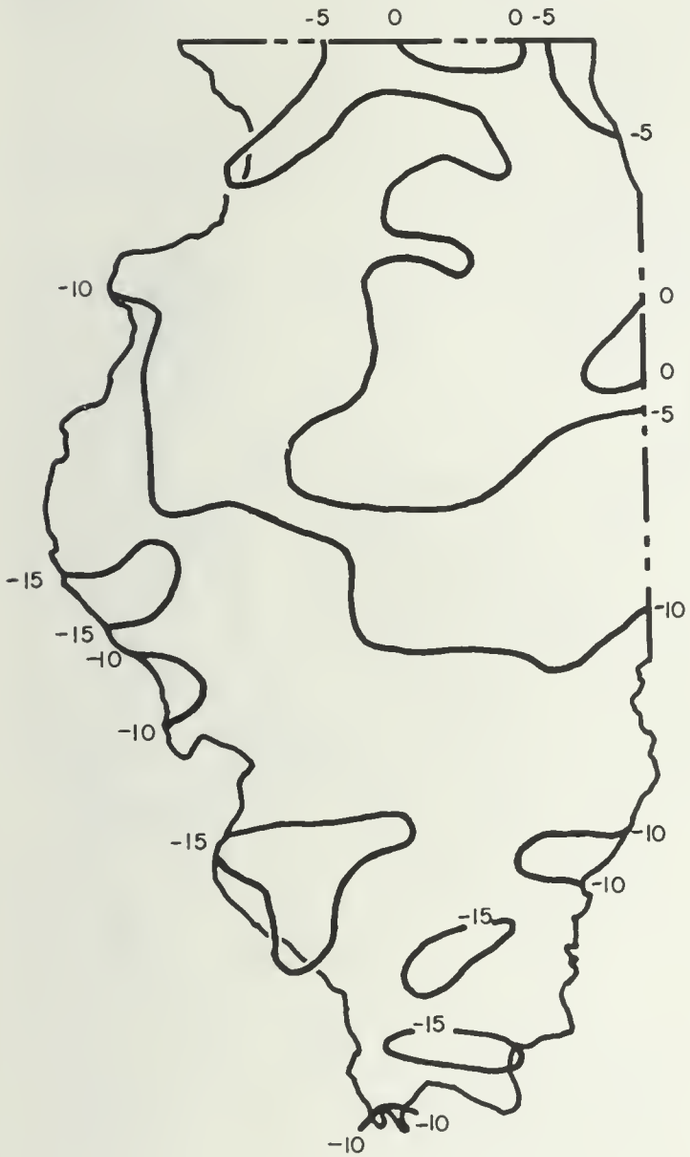


FIGURE 9. RAINFALL DEPARTURE FROM NORMAL IN INCHES. January to December 1953.

Rainfall Deficiency Patterns. Figure 8 shows the rainfall deficiency pattern for the period April through December 1952; it reveals that the southern half of the State had deficiencies of 5 to 10 inches. Figure 9 shows the rainfall deficiency pattern for 1953, for which the deficiency was 10 to 17 inches throughout southern Illinois. In addition, an area in the western part of the State experienced rainfall deficiencies of 10 inches or more.

The precipitation pattern for 1954 was similar to those of the previous years in that deficiencies again appeared in the southern half of Illinois (Figure 10). The greatest deficiency reported was at Windsor, Shelby County (14 inches), and deficiencies were relatively small in the area south of Route 50, which runs from East St. Louis across the state to Lawrenceville. There was only slight deficiency in the Randolph-Perry County area. Above-normal rainfall occurred in a large part of northern Illinois, especially in the Chicago region, where the excess was largely due to the storm of October 1954[15].

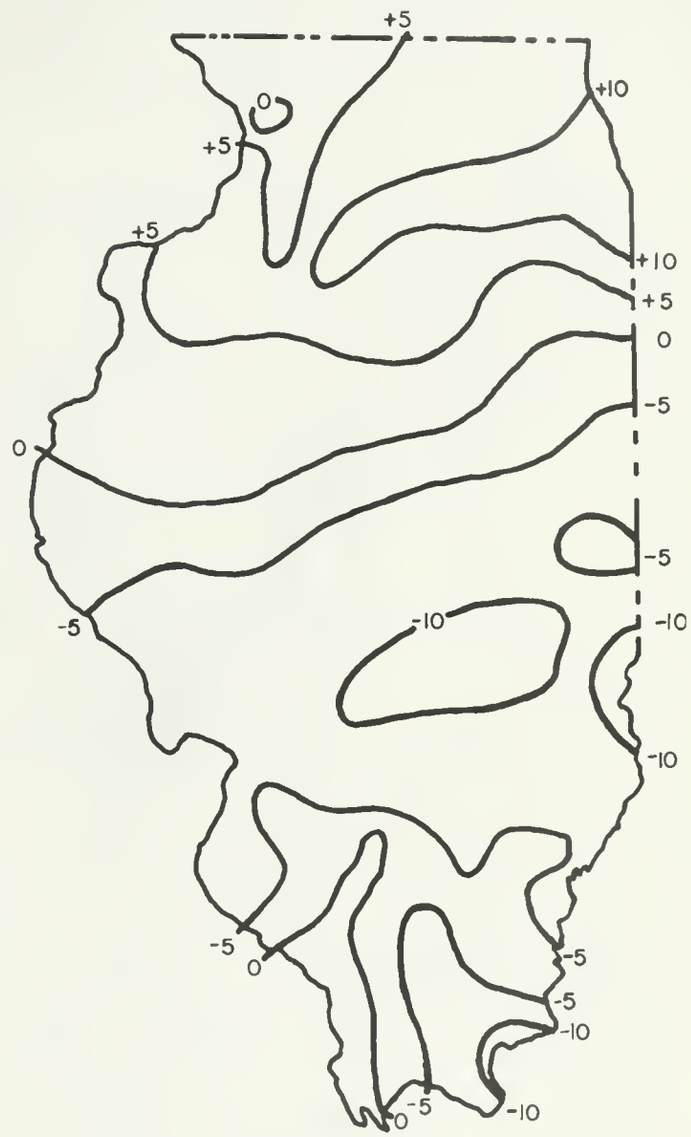


FIGURE 10. RAINFALL DEPARTURE FROM NORMAL IN INCHES. January to December 1954.

Figure 11 is a graph of the rainfall trends during the period 1952-55, showing the cumulative

departures from normal rainfall for each of five stations. Three of these stations, Quincy, Taylorville, and Urbana, are in the northern part of the drought zone, while the Centralia and East St. Louis stations represent more nearly the central part of the drought zone. From this figure it will be seen that there was an improvement in conditions at Quincy commencing about the middle of 1954. This was followed later in 1954 by a rapid improvement in conditions at Centralia. A less marked improvement took place at East St. Louis later in 1954. The slopes of the cumulative departure curves for Taylorville and Urbana reduced during 1954, but no marked recovery was indicated by the data.

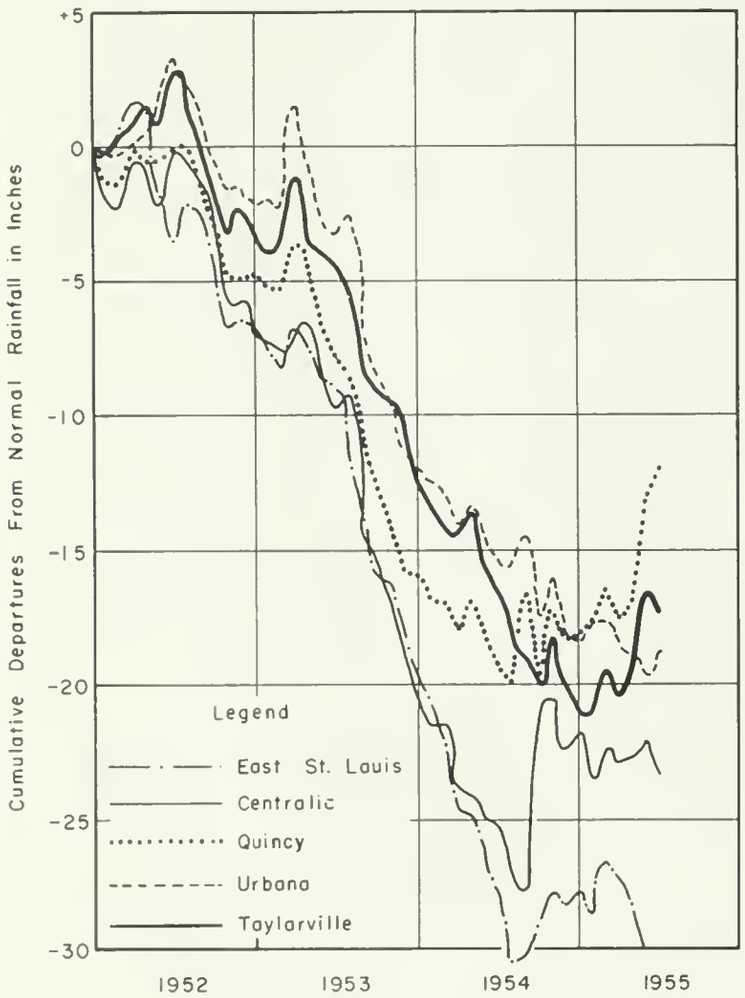


FIGURE 11. CUMULATIVE RAINFALL DEPARTURES FOR 1952 to 1955.

Runoff Conditions

Early in 1954 arrangements were made with the U. S. Geological Survey to expedite reporting of current runoff data from certain key stream-gaging stations in the southern half of Illinois. These data were combined with earlier runoff data to produce cumulative 12-month maps of runoff conditions, prepared each month until precipitation relieved the situation in the fall of 1954. The information reported by the U. S. Geological Survey consisted of provisional runoff values that are subject to revision when computation for publication is completed. These provisional data have, however, been used throughout this report for the entire period commencing in October 1953. Data prior to that date were final and not subject to revision. The watersheds for the 15 stations used are shown in Figure 12.

FIGURE 12. DRAINAGE AREAS USED IN STREAMFLOW MAPPING

1. LaMoine River at Ripley
2. No. Fork Mauvaise Terre Ck. near Jacksonville
3. Salt Ck. at Greenview
4. Sangamon River at Monticello
5. Embarrass River at Ste. Marie
6. Kaskaskia River at Shelbyville
7. So. Fork Sangamon River at Kincaid
8. Macoupin Ck. near Kane
9. Shoal Ck. near Breese
10. Kaskaskia River at New Athens
11. Little Wabash River at Wilcox
12. Skillet Fork at Wayne City
13. Little Wabash River at Carmi
14. Big Muddy River at Plumfield
15. Cache River at Forman

There is some overlapping of the information supplied from these stations. Shoal Creek at Breese

and the Kaskaskia River at Shelbyville are both tributary to the area measured by the Kaskaskia River gage at New Athens. Little Wabash River at Wilcox and Skillet Fork at Wayne City are both subdivisions of the drainage area of the Little Wabash above Carmi.

An additional factor to take into account in considering the runoff data was the substantial pumpage of groundwater into the Kaskaskia River near Bondville. In mapping the runoff, correction was made by subtracting the groundwater pumpage at Bondville (minus losses due to industrial use at Tuscola) from the discharge at both Shelbyville and New Athens. In order to make the data more representative for sub-areas, such as the Kaskaskia drainage basin below Shelbyville, the weighted effects of runoff at Shelbyville and at Shoal Creek at Breese were subtracted, so that the runoff for the Kaskaskia River basin exclusive of these two drainage areas could be computed. Similar corrections were made for the Little Wabash River basin. While it would have been desirable to make adjustment for sewage discharge into the various streams, this was not done. On certain streams such correction could be a significant factor during extreme low-flow periods.

TABLE 8

COMPUTATION OF MOST EXTREME LOW-FLOW PERIOD OF 12 MONTHS DURATION (1953-54 DATA)

Gaging Station	Streamflow in Inches for the 12-Month Period Through: April	May	June	July	Aug.	Sept.
Big Muddy River at Plumfield	1.64	.65	.73	.72	.77	.96
Embarrass River at Ste. Marie	1.57	.76	.57	.39	.36	.35
Kaskaskia River at New Athens	1.28	.69	.59	.38	.38	.51
Little Wabash River at Carmi	1.51	.48	.44	.40	.48	.66
Little Wabash River at Wilcox	.97	.37	.29	.24	.26	.43
Macoupin Creek at Kane	.87	.60	.22	.20	.23	.32
Mauvaise Terre Creek at Jacksonville	.58	.25	.08	.08	.22	.23
Shoal Creek near Breese	1.74	.96	.41	.33	.35	.49
Skillet Fork River at Wayne City	1.24	.48	.52	.53	.60	.91
Sangamon River at Kincaid	.98	.70	.52	.42	.42	.43
Kaskaskia River at Shelbyville	1.94	1.40	1.28	.74	.77	.78
Average	1.30	0.67	0.51	0.40	0.44	0.55

For mapping the runoff, the data (expressing discharge in inches) were entered in the center of the appropriate drainage areas. Contours were then drawn (Figures 13-18) on the basis that each runoff value represented the mean of the area, and that discharge within a gaged area was not necessarily geographically uniform. For example, streamflow for the 12 months through July 1954 for the Embarrass River at Ste. Marie was 0.39 inches. By comparison of Figures 14 and 12 it may be seen that several contours cross the drainage area of the Embarrass. In drawing the contours it was attempted to locate them so that the mean runoff for the drainage area would be 0.39 inches. It appeared from the data available that runoff was less than 0.2 inches in part of the basin and more than 0.6 inches in another part.

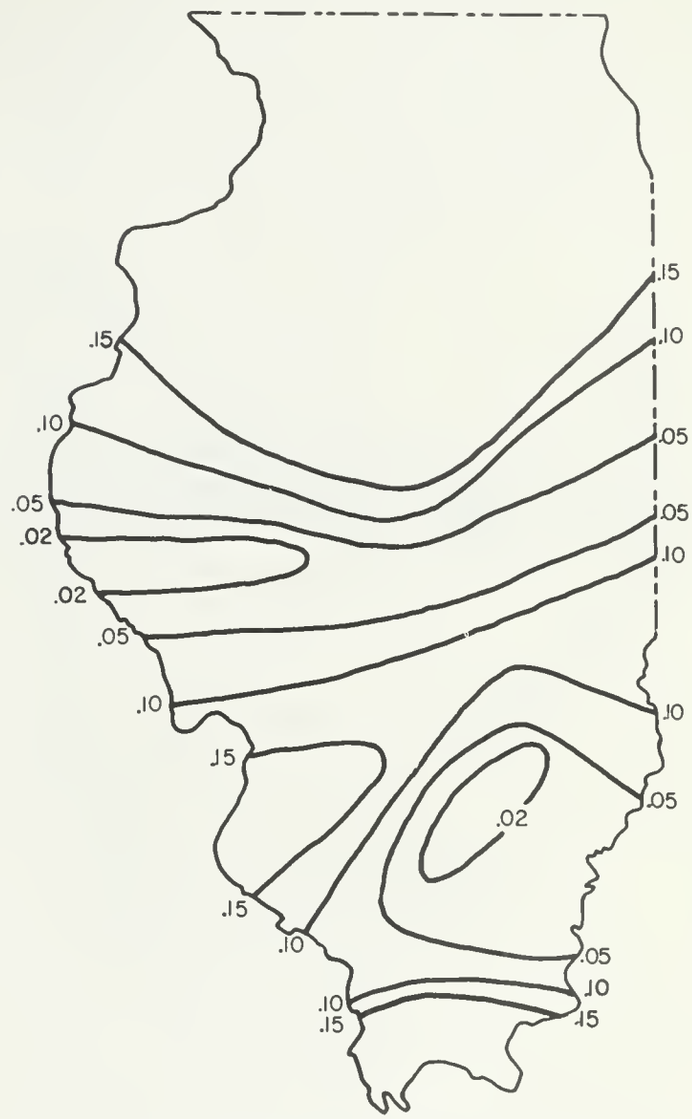


FIGURE 13. RUNOFF IN INCHES - 6 MONTHS ENDING JANUARY 1954.

Periods of Lowest Streamflow. For each map used in charting the progress of the drought, 12-month sliding totals of runoff from all of the 15 areas were prepared. Subsequently one map was drawn for each of the 6, 12, 18, 24, 30 and 36-

month duration periods. The particular low-flow period for each of these durations was selected by averaging, by calendar months, the sliding totals for these gaging stations, except that the Sangamon River at Monticello, Cache River at Forman, Salt Creek at Greenview, and La Moine River at Ripley were excluded as being outside the principal area of drought.

Table 8 illustrates the method of computation of the most extreme of the low-flow periods of 12 months' duration. It will be seen from this table that the lowest average of the 11 stations occurred for the 12 months ending with July 1954.

Estimates of runoff computed from impounding reservoir records at Springfield, Waverly, Staunton, Nashville, DuQuoin, Carbondale, West Frankfort, Johnston City, McLeansboro, Norris City, Eldorado, Mattoon, and Oakland were also used in drawing the maps so far as the available data permitted the making of estimates. For the most part, these did not contribute usable data because of the lack of periodic readings of reservoir levels during 1953 and earlier. Qualitative guidance in the preparation of the contour maps was obtained, however, from observations made on the reservoirs during each month.

Figure 13 shows the streamflow pattern for the 6-month period during which the average of the runoff at the 11 stations considered to be within the drought area was the lowest in 1952-55. This 6-month period terminated January 31, 1954. Figure 14 shows the runoff picture for the lowest 12 months, which ended July 31, 1954. Figure 15 shows the total runoff for the 18 months ending December 31, 1954, the lowest 18 months' period found. Figures 16, 17, and 18 show respectively the lowest 24, 30 and 36 months' totals found. No runoff data later than May 1955 were available at the time these maps were prepared.

These maps depict the streamflow patterns for periods in which low runoff occurred over large areas within the current drought. In several cases,

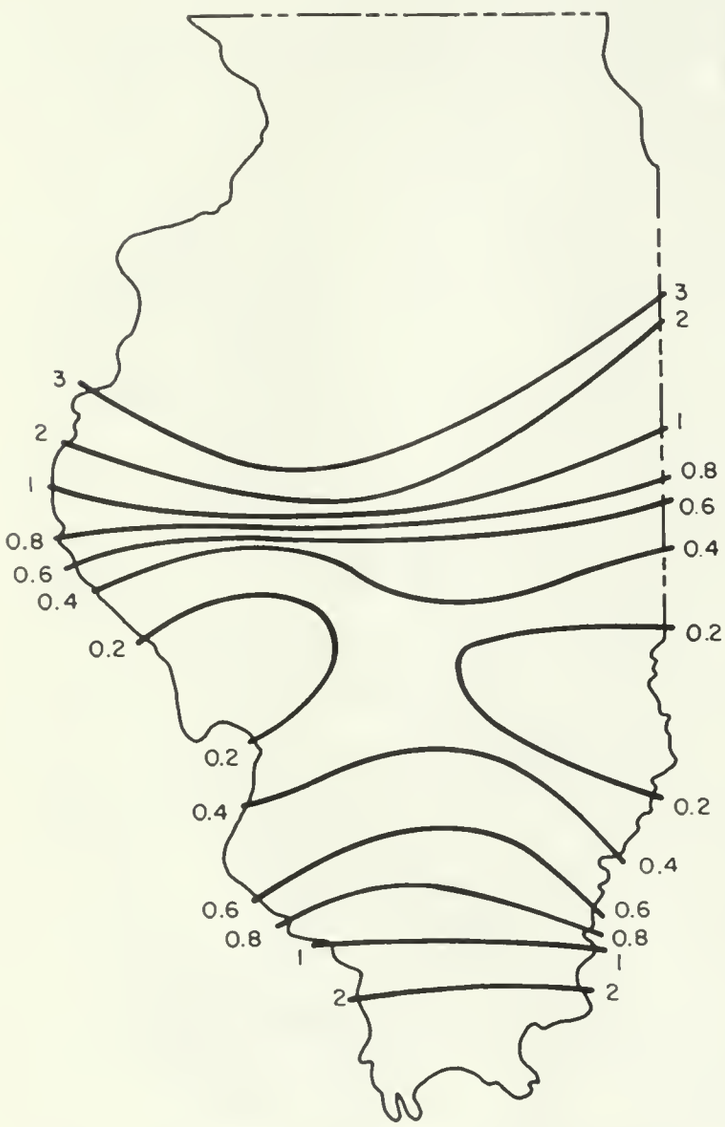


FIGURE 14. RUNOFF IN INCHES - 12 MONTHS ENDING JULY 1954.

FIGURE 15. RUNOFF IN INCHES - 18 MONTHS ENDING DECEMBER 1954.

values as low as those indicated on the maps occurred over considerable areas during other periods. For example, for the 12-month period ending June 30, 1954, run-off in Pike, Scott, Greene, and in the southern portions of Morgan, Sangamon and the north part of Macoupin counties was 0.20 inches or less. Runoff during this period for Mauvaise Terre Creek at Jacksonville was 0.11 inches. A few anomalous values were obtained from reservoir records, as for example at Mattoon, where runoff in 1954 was less than evapotranspiration along the waterways and in the reservoir bottom. During this period Mattoon was unable to pump any water from its lake, and relied upon groundwater sources.

For the purposes of this report, the maps have been prepared on the assumption that unit runoff is independent of drainage basin area. This may be incorrect, but the relationship between these two factors has not been established[16], and requires extensive additional study. It would seem reasonable to expect smaller low-flow values from a small basin than from a large one. A study to evaluate this effect is of importance for safe and economical design of water works.

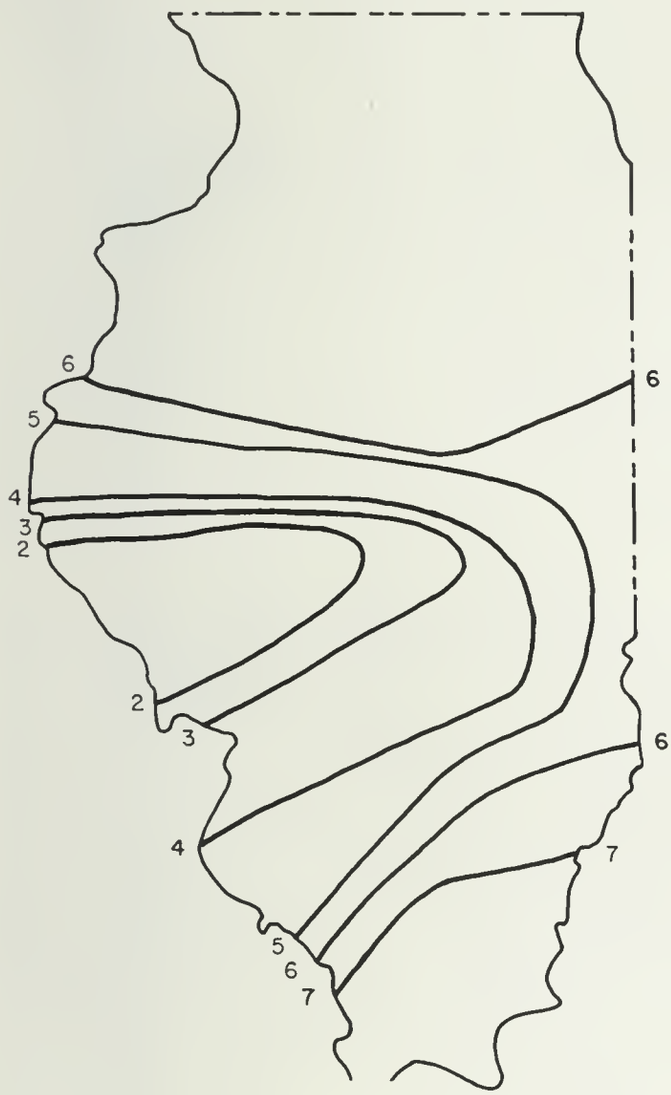


FIGURE 16. RUNOFF IN INCHES - 24 MONTHS ENDING DECEMBER 1954.

FIGURE 17. RUNOFF IN INCHES - 30 MONTHS ENDING DECEMBER 1954.

Drought Frequency Studies

Streamflow records in Illinois extending back prior to 1915 are available for Little Wabash River at Wilcox, Embarrass River at Ste. Marie, Big Muddy River at Plumfield, Kaskaskia River at Vandalia, Sangamon River at Riverton, Sangamon River at Monticello, Spoon River at Seville, and Pecatonica River at Freeport. Four of these stations began operating in 1908 or 1909, but all of them were interrupted in 1912-14. In addition, records are available beginning in 1921 for LaMoine River at Ripley and Macoupin Creek near Kane (although the records for 1934 to 1939 were estimated for the latter), and similar data are available for Mackinaw River near Green Valley. These records were examined and, because of manpower limitations, it was decided to limit runoff frequency studies to five long-record stations in or near the drought region.

Period of Record. The period chosen for use was the longest for which records were available in the area and generally covered the 41 years, 1914 to 1955. Some of the streamflow data were available beginning in 1908, and some did not begin until as late as 1921. The specific periods used for the determinations of frequency are given elsewhere, but there may be question as to whether the period 1914 to 1955 was a representative sample in time.

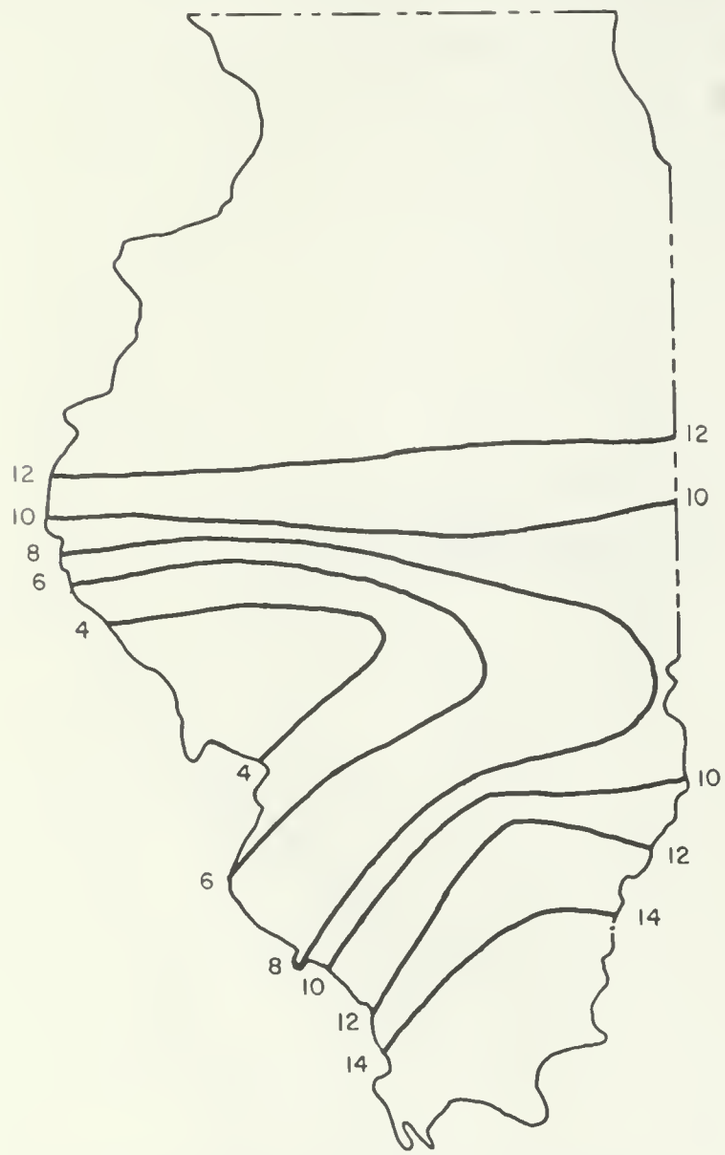


FIGURE 18. RUNOFF IN INCHES - 36 MONTHS ENDING APRIL 1955.

Rainfall data were available from 1906 at four of the five stations studied. A determination of frequency of low-rainfall periods by methods described below for Springfield, Quincy, Olney, Mt. Vernon, and Urbana, indicated that the values of rainfall for the 48-year period 1906-54 checked within 4 per cent those for a 40-year period of 1914-54 for 20-to 40-year frequencies.

The longest rainfall record available in the area was that for St. Louis, Missouri, which extends from 1837 to the present. Frequency determination based on 12- and 24-month low rainfall periods for the St. Louis gage indicated that the period 1914-54 generally gave values of rainfall 5 to 10 per cent lower than those for corresponding recurrence intervals for the period 1837 to 1954. Accordingly an attempt was made to check the St. Louis rainfall record against other long-time records by double-mass correlations. The nearest stations having records of comparable length were those for Cincinnati and Portsmouth, Ohio. Double-mass correlations of these stations against the St. Louis record, and against each other, indicated changes in slope attributable to the St. Louis record in the years 1845 and 1860, and the data indicate that the rainfall values recorded for St. Louis between these years may have been as much as 20 per cent too high. It was felt that it would be necessary to make an investigation of locational factors at all three stations, with special attention paid to changes in the St. Louis gage at about 1845 and 1860. Adjustment of high values for St. Louis would have brought the frequency determinations for the period 1837-1954 into closer comparison with the period 1914-54. Since it was not possible to study the representativeness of the period more completely without study of the histories of the St. Louis, Cincinnati, and Portsmouth gages, it was decided to treat the data available as though the period of record from 1906 to 1955 was a representative period.

Computing Sliding Totals. Stations chosen from intensive drought-frequency study were Big Muddy River at Plumfield, LaMoine River at Ripley, Little Wabash at Wilcox, Macoupin Creek at Kane, and Sangamon River at Monticello. These five gave reasonably good geographic coverage of the drought area and had periods of record of 34 to 45 years (See Table 4). Published runoff data are available for all of these stations from their inception to September 30, 1953. Provisional data were used for the period October 1, 1953 through May 1955. From these data, recorded as inches of runoff per month, sliding totals were computed for periods of 6, 12, 18, 24, 30, 36, 48, and 60-month duration for each of the five streams.

Rainfall data from five United States Weather Bureau Cooperative rain-gaging stations were similarly tabulated and sliding totals prepared for the 12- and 24-month periods. The rainfall stations used for the study were: Effingham, Mount Vernon, Olney, Springfield, and St. Louis. All but Effingham were for the period 1906-1955.. The Effingham data were for 1918-55. In addition to the 12- and 24-month totals, 6, 18, 30 and 36-month totals were prepared from the St. Louis data.

In the preparation of this study, thought was given to the propriety of arraying all of the n-month totals of runoff, for determination of frequency. It appeared to the writers that this would be subject to possible error since many of the extreme values are related to each other. It was concluded

that each drought or low-flow period is a separate event, properly characterized by the lowest n-month total, and that the inclusion in the array of other totals occurring within the most extreme "n" months would produce misleading probability determinations. This does not agree with the approach taken by others, notably Thompson[43]. To carry the writers' conclusion in this regard into effect, particular pains were taken to establish procedures that would make each extreme value independent of all others.

From each table of sliding totals, the lowest values were selected. The procedure used in making these selections was designed to eliminate dependence between values used. The procedure was as follows: First, the lowest value for the entire period of record was determined by scanning, and marked. Adjacent values prior to and following the chosen value were then "lined out" for a number of months equal to the duration period under study, less one month. This procedure manually cancelled these values from further consideration. The next lowest value was then determined by scanning, and adjacent values similarly excluded. This procedure was continued until a number of values approximately equal to one-half of the period of

Table 9

12-MONTHS RUNNING TOTALS OF RUNOFF IN INCHES

Little Wabash River At Wilcox, Illinois

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
1915							7.74 16	10.91	11.11	11.03	11.22	12.38
16	15.95	15.77	16.02	16.15	15.26	14.19	12.63	9.61	9.32	9.27	9.08	8.03
17	5.14	3.25 6	3.46	4.52	6.30	7.81	7.83	7.74	7.74	7.77	7.82	7.71
18	6.80	8.01	7.63	8.68	8.98	6.87 15	7.48	7.56	9.01	9.11	9.37	12.03
19	12.63	11.97	13.14	10.98	9.69	10.88	10.56	10.42	8.97 20	9.63	11.77	10.48
1920	10.02	10.80	11.99	13.34	14.22	13.11	12.91	13.18	13.40	12.62	10.18	8.07
21	9.53	8.60	7.92	7.33	5.80	5.63	5.49	5.44 11	5.48	5.59	6.48	7.57
22	7.14	6.99	7.46	10.55	10.61	10.63	10.70	10.46	10.19	10.07	9.17	7.99
23	8.03	8.25	8.34	5.43 10	6.17	6.52	6.61	7.27	7.42	7.65	7.83	9.81
24	10.32	11.20	9.55	9.28	9.20 21	10.35	10.23	11.78	9.83	9.62	9.51	8.49
25	7.97	7.35	6.49	5.80	4.83	3.45	3.46	3.29	3.10 5	3.14	3.79	4.32
26	3.66	4.30	5.73	7.56	7.60	7.43	7.39	7.41	8.34 17	9.84	10.31	10.08
27	11.58	11.55	12.48	13.42	15.42	16.71	16.75	17.04	16.24	16.00	15.10 30	17.72
28	17.02	17.85	16.27	14.36	12.71	13.07	13.30	13.35	13.24	12.21	12.31	10.61
29	11.44	9.68 22	10.71	12.07	15.40	14.94	15.09	14.87	14.86	14.61	14.53	14.90
1930	16.70	17.61	16.41	14.51 29	10.83	9.61	9.14	8.95	9.30	9.30	9.00	7.34
31	3.31	1.94	1.79	1.49 3	1.54	1.57	1.79	1.95	2.36	2.48	3.59	5.38
32	10.29	10.70	10.33	10.35	10.30	10.59	10.43	10.38	9.96	9.83	9.27	8.45
33	6.42 13	6.55	9.62	12.26	16.49	14.28	16.23	16.19	15.90	15.88	15.32	14.28
34	11.43	10.78	7.93	5.56	1.29	1.21 2	1.63	2.11	2.41	2.64	4.41	5.51
35	7.33	7.79	10.27	10.42	14.88	15.21	14.95	14.41 28	14.05	13.81	12.39	11.37
36	9.80	10.46	3.02	9.04	4.74	4.38	4.23	4.22 9	4.23	4.25	4.49	4.46
37	8.62	9.39	8.62	9.20	10.72	11.15	11.23	11.28	11.29	11.50	10.93	11.92
38	7.66	6.66 14	9.55	11.90	10.76	10.62	11.27	11.42	11.46	11.22	11.33	9.38
39	10.61	12.14	11.03	10.06 24	9.61	10.14	9.47	9.82	9.78	9.78	9.65	9.56
1940	9.20	7.09	5.00	4.01 8	4.90	4.22	4.14	3.61	3.60	3.63	3.68	3.81
41	3.92	3.67	3.34	2.46	1.54 4	2.08	2.12	2.13	2.18	2.43	3.87	4.07
42	4.13	6.34	8.37	9.11	9.42	10.03	12.30	12.50	12.46	12.19	11.72	12.15
43	12.30	10.69	9.51	8.82 18	18.75	16.90	15.58	15.44	15.43	15.43	15.43	13.67
44	13.25	12.94	12.67	15.65	5.74	5.79	5.61 12	5.71	5.80	5.80	5.77	5.77
45	5.84	5.96	10.01	11.11	12.41	16.22	16.73	16.73 31	17.05	17.71	18.37	19.12
46	20.64	22.74	18.76	14.04	17.69	14.13	13.82	14.11	13.72	13.10 25	13.46	13.57
47	13.24	11.83	11.35	13.74	9.93	10.01	10.25	9.94	10.21	10.42	9.60	8.90 19
48	9.70	9.79	12.63	11.82	10.95	10.61	11.06	11.04	10.80	10.63	12.16	13.11 26
49	18.06	19.48	18.13	17.12	16.77	16.86	16.42	16.42	16.53	18.63	16.99	18.10 32
1950	20.24	22.24	22.25	23.28	23.40	24.24	24.27	24.28	24.50	22.36	22.98	21.57
51	14.35	13.65 27	14.03	13.81	13.50	12.47	12.77	12.72	12.42	12.51	12.64	12.72
52	12.17	9.69 23	10.58	11.48	11.44	11.78	11.07	11.04	10.98	10.84	10.00	9.13
53	7.73	6.40	5.04	3.46	3.99	3.46 7	3.46	3.45	3.44	3.44	3.43	3.40
54	3.38	3.32	1.71	0.97	0.37	0.29	0.24 1	0.26	0.43	0.61	0.62	0.75
55	1.02	2.38	3.42	3.60	3.78	4.30						

record divided by the duration period (expressed in years) was obtained. Choice of this number of values was arbitrary, and yielded sufficient terms to define the frequency curves satisfactorily. The values thus selected were then arrayed in order of magnitude.

The procedure used is illustrated in Table 9, which shows the 12-month sliding totals for the Little Wabash River at Wilcox for the period 1914-1954. Inspection of the data in Table 9 revealed that the 12-month period of lowest flow occurred for the period ending with July 1954. Accordingly

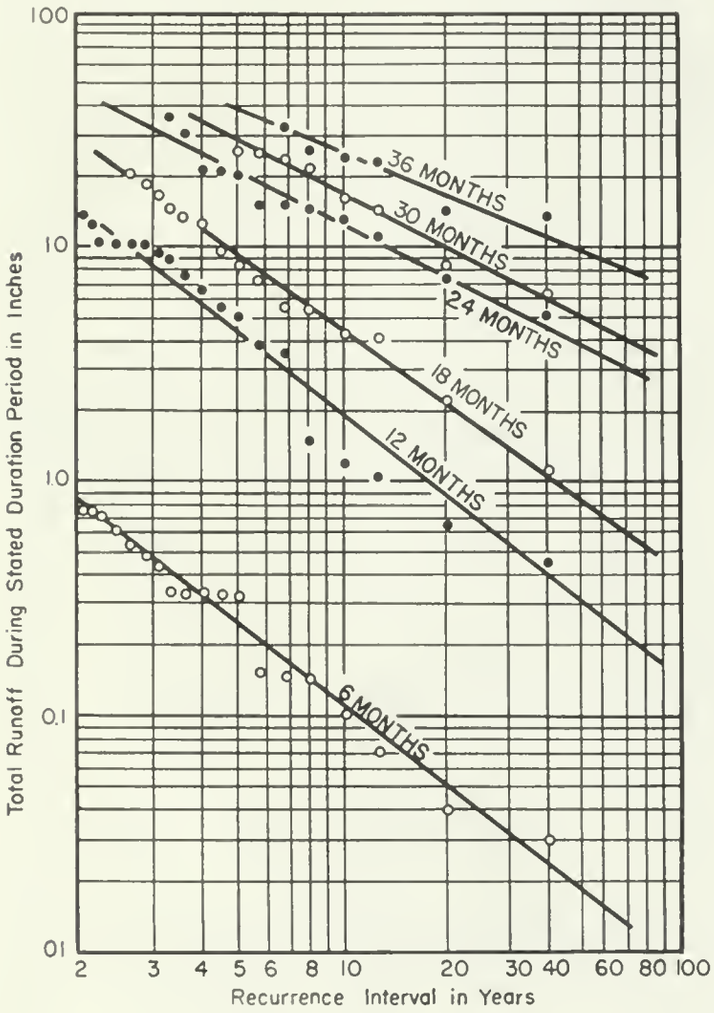


FIGURE 19. FREQUENCY OF LOW FLOW OF VARIOUS DURATIONS. Big Muddy at Plumfield.

the 11 values preceding and succeeding this month were ruled out as shown and the critical value was marked with the rank number "1". Next-lowest value in the table was found to occur in June 1934, and a similar procedure of exclusion was followed. This procedure was repeated until values exceeding the mean streamflow were reached. In the earlier stages of the work, it was attempted to commence at one end of the table and work toward the other, but it was found that this resulted occasionally in the choice of less extreme values and allowed some subjective error. Therefore the procedure of choosing successively the most extreme remaining values was adopted.

After the manual computation and arraying described above was completed, monthly runoff values on key-punch cards became available. The use of the key-punch cards for this operation greatly speeded the work, but was subject to some limitations. For the conduct of the operation by punch cards, it was found most convenient to program the sorting operation so that it produced a deck of cards containing the sliding totals for the duration period under study. These were then arrayed in order of magnitude by machine sorting, the rank numbers were punched on the cards, the cards then re-sorted into chronologic order, and a printed tabulation of the sliding totals

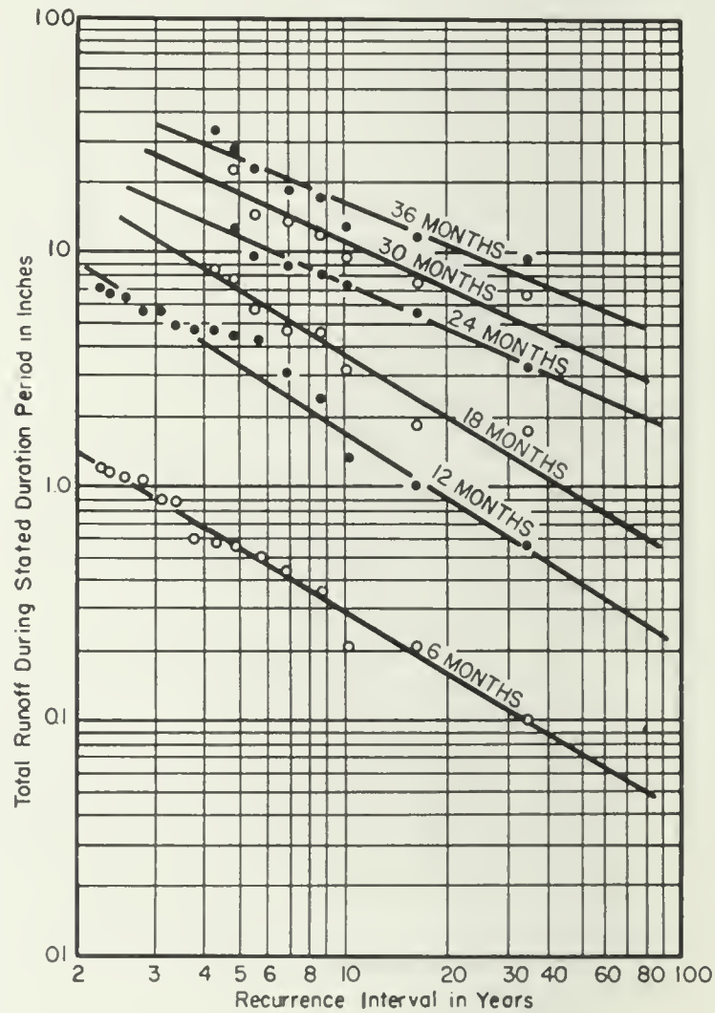


FIGURE 20. FREQUENCY OF LOW FLOWS OF VARIOUS DURATIONS. LaMoine River at Ripley.

and rank numbers was prepared. No mechanical method for screening the values for independence was found, and it was necessary to choose values by inspection. For the determination of rank numbers out of each series of nearly 500 values, only the most extreme 100 values were arrayed and rank numbers printed. This procedure reduced the scanning required.

Plotting Positions. Study was given to the determination of plotting position. Methods have been proposed by Allen Hazen[17], E. J. Gumbel[18], C. S. Jarvis, et al[19] and others. The "California" method[20] was used as being the simplest and

apparently as meritorious as any of the others. With this method, the recurrence interval (R) is expressed as follows:

$$R = \frac{T}{m}$$

in which T is the length of record in years, and m is the rank number of the event when the events are arrayed in order of magnitude. For a 40-year record, the extreme event therefore would have a recurrence interval of 40 years, the second event - 20 years, the third - 13.33 years, etc.

In the last few years advanced methods for analysis of probability of extreme events have been developed. Chow[21] has observed that, in many cases, hydrologic data follow a log-probability law, under which the logarithms of hydrologic events are found to correlate well against frequency parameters. Gumbel has also made use of the logarithms of hydrologic events in studying drought probabilities[22]. Gumbel had not, however used a general definition of drought. He considered a drought to be the day of minimum discharge for a stream. Apart from the work of Hazen[17] [23], little published work is known in the field of low-flow duration.

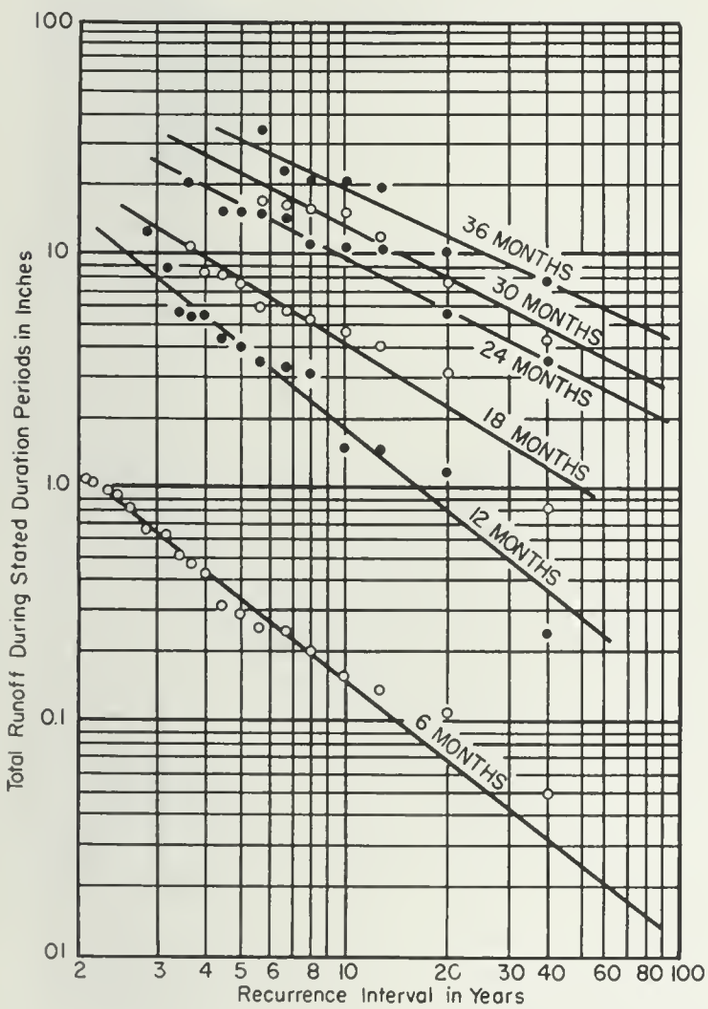


FIGURE 21. FREQUENCY OF LOW FLOWS OF VARIOUS DURATIONS. Little Wabash at Wilcox.

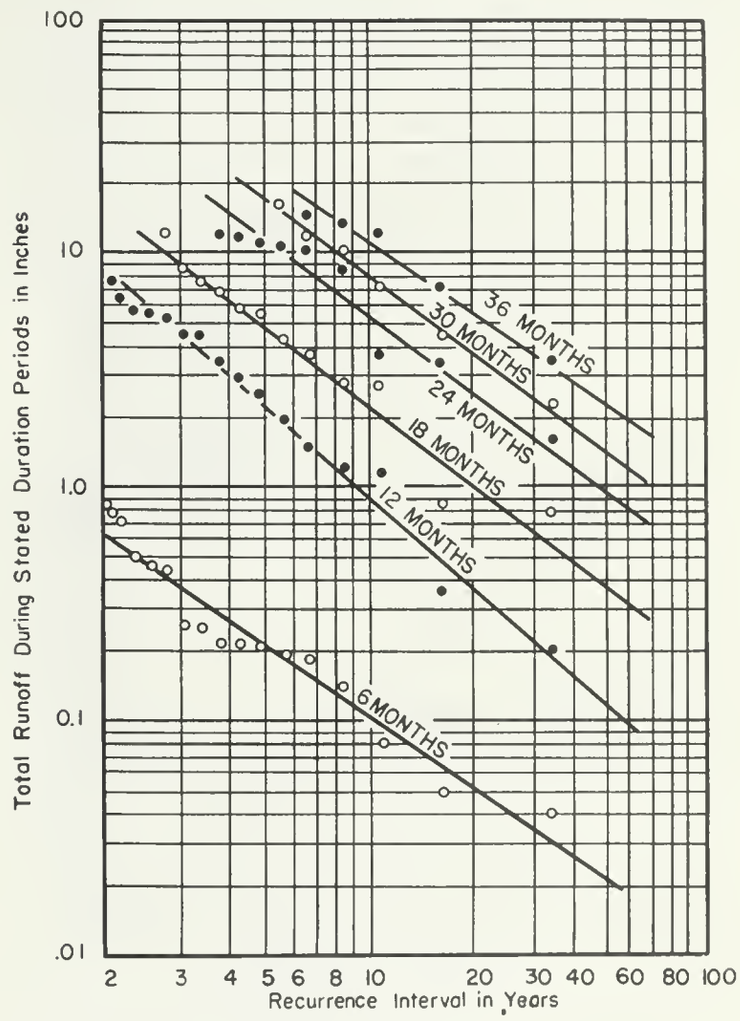


FIGURE 22. FREQUENCY OF LOW FLOW OF VARIOUS DURATION. Macoupin Creek at Kane.

Stating the recurrence interval in years does not imply that an event having a recurrence interval of "R" years will happen every "R" years. It means instead that, in a period of time of length "n" much longer than "R", such an event would be expected to happen $\frac{n}{R}$ times. Thus a period of low steamflow having a recurrence interval of 100 years would be expected to be equalled or exceeded (in duration) 10 times in 1000 years. Obviously any estimates of recurrence interval values that are greater than the length of the period of record are extrapolated and must be regarded as approximations.

Graphic Determination of Frequency and Duration of Low Flows. Preliminary graphical analyses were made by several different methods. The hydrologic data were plotted on arithmetic extreme-probability paper, and showed pronounced curvature. Curvilinear relationships were also noted in semi-logarithmic and logarithmic plots. Good approximations of straight lines appeared to occur when the extreme-probability paper designed by Gumbel was used in conjunction with logarithms of hydrologic data. The data for the five stream-gaging stations analyzed in this study are shown on extreme probability-logarithmic

graphs in Figures 19 to 23. Each figure shows the data for six of the eight duration times considered. The 48- and 60-month data were omitted from these graphs but are presented in a subsequent figure.

Close examination of the data in Figures 19-23 indicates that, for any given recurrence interval, there are noticeable differences between runoff values for the 5 basins which may be related to drainage area, climatic differences, groundwater conditions, or other hydrologic data.

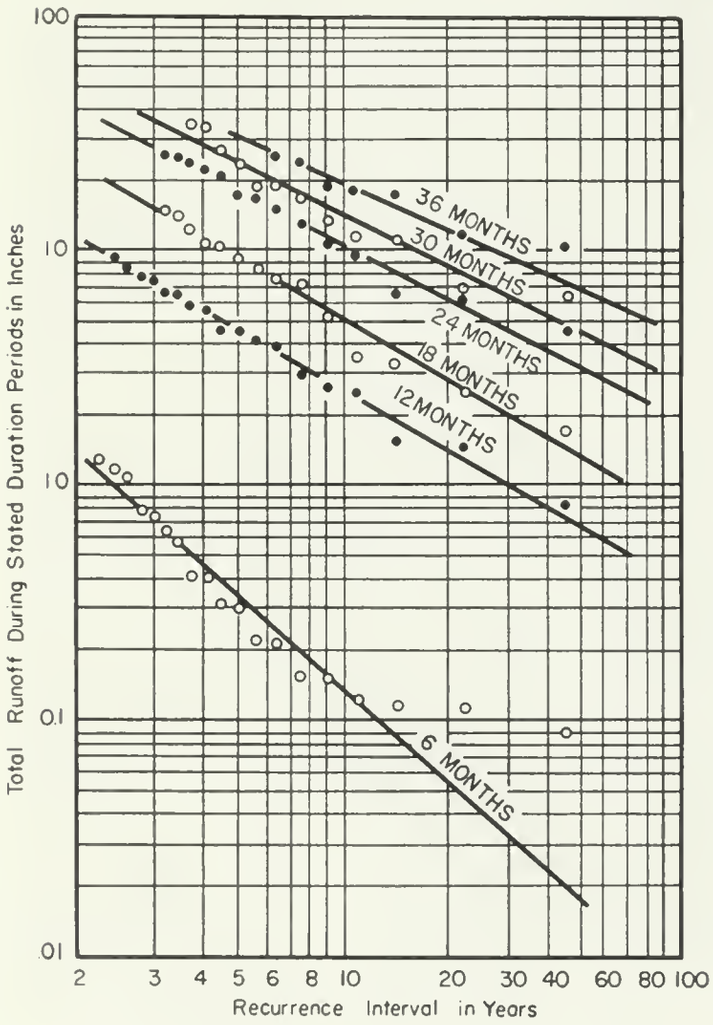


FIGURE 23. FREQUENCY OF LOW FLOW OF VARIOUS DURATIONS. Sangamon River at Monticello.

Families of curves for the various duration periods were drawn by eye in Figures 19-23. Since no systematic curvature was indicated by the data, all were drawn as straight lines. It will be noted that the curves for 12, 24, and 36 months appear to form a reasonable series for each station, but that the curves for 6, 18, and 30 months do not seem to correspond to such a series. Instead of drawing all the curves in each figure as members of one family, it was assumed, in drawing Figures 19 to 23, that the curves for 6, 18, and 30 months could comprise a separate series from the remaining curves, since the 6, 18, and 30 months' curves cover fractional parts of the annual hydrologic cycles. The appearance of these separate families of curves deserves further study; it is not considered that the data conclusively establish the presence of systematic differences, however seasonal variances suggest that such differences might be expected.

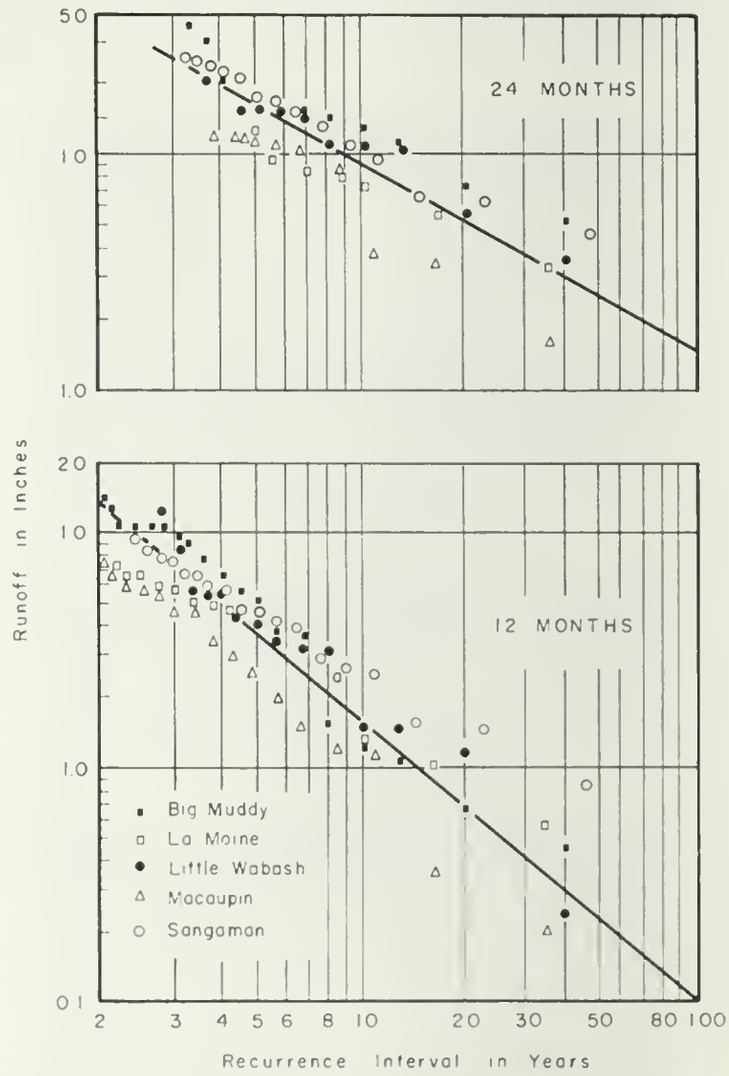


FIGURE 24. LOW FLOW FREQUENCY DATA CONSOLIDATED ON BASIS OF FLOWIN INCHES.

It should be evident that, in a record of 30 to 45 years' duration, very few low-flow periods of three-to-five year duration will occur. Because of this, it is felt that the estimates of recurrence interval for low-flow periods of three years or more, are not altogether dependable. In some cases frequency curves had to be based on very few values. The situation is somewhat improved by consolidating the data from five stations on one graph, as in Figure 24 but it still appears possible that the straight lines drawn may not accurately represent probable frequencies over a very long period. If the period of record were in ex-

cess of 100 years, more reliance might be placed on these determinations.

Graphs were made of the runoff in inches versus recurrence interval for each duration period, combining the data from the five stations. These showed some scatter of the data (See Figure 24).

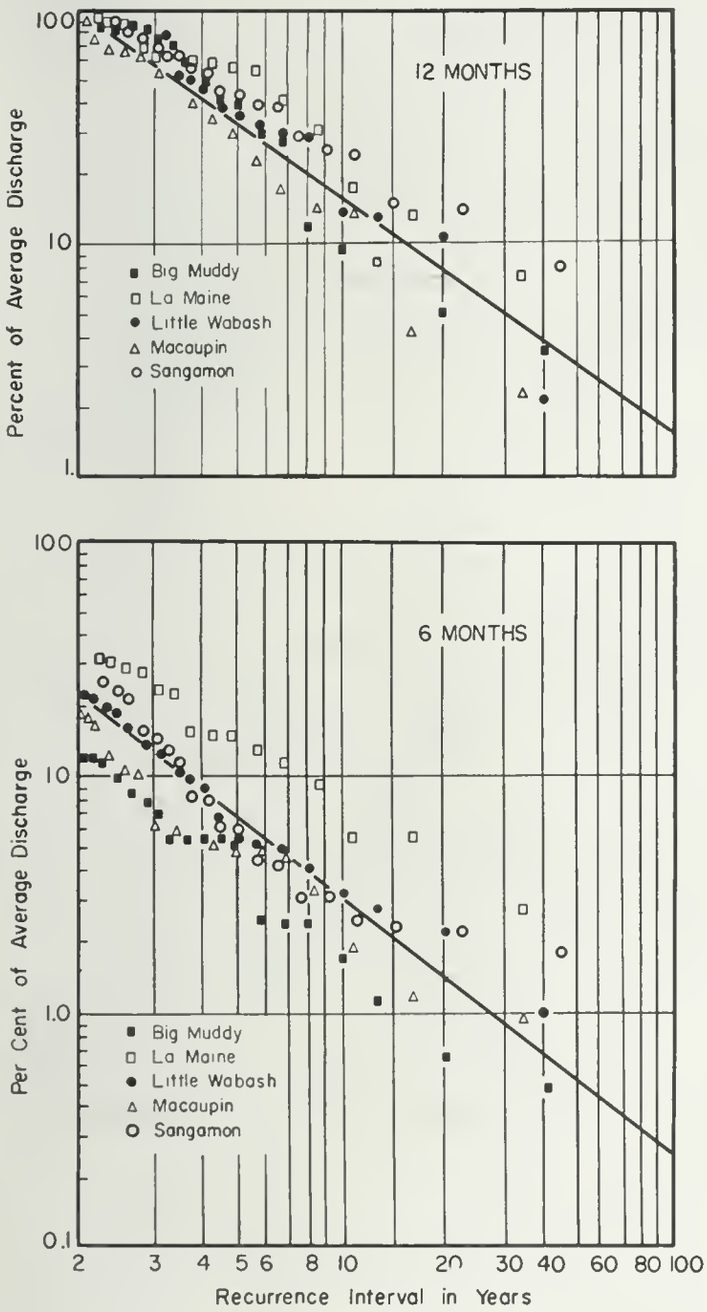


FIGURE 25. LOW-FLOW FREQUENCY DATA, CONSOLIDATED ON BASIS OF PER CENT OF AVERAGE FLOW.
6- and 12-Month Duration.

Seeking a better way of coordinating the data from the various stations, plottings were made of the per cent of average discharge rate versus recurrence interval. Average rate data are given in Table 4. These graphs showed better correlations of the data, especially for the more-numerous less-extreme events, than were obtained when the absolute amounts of runoffs were used. These unified plottings are shown in Figures 25-28.

Close inspection of the data in Figures 25-28 indicates that there may be significant differences in slopes of the lines for each particular station. For example, the slope of the 12-month curve (Fig. 25) which might be drawn for the Sangamon River appears to be smaller than that which might be drawn for Macoupin Creek. The differences between these lines may be due to sampling errors or they may be due to hydrologic differences between the areas gaged which cause differences in variability and in relative runoff between streams.

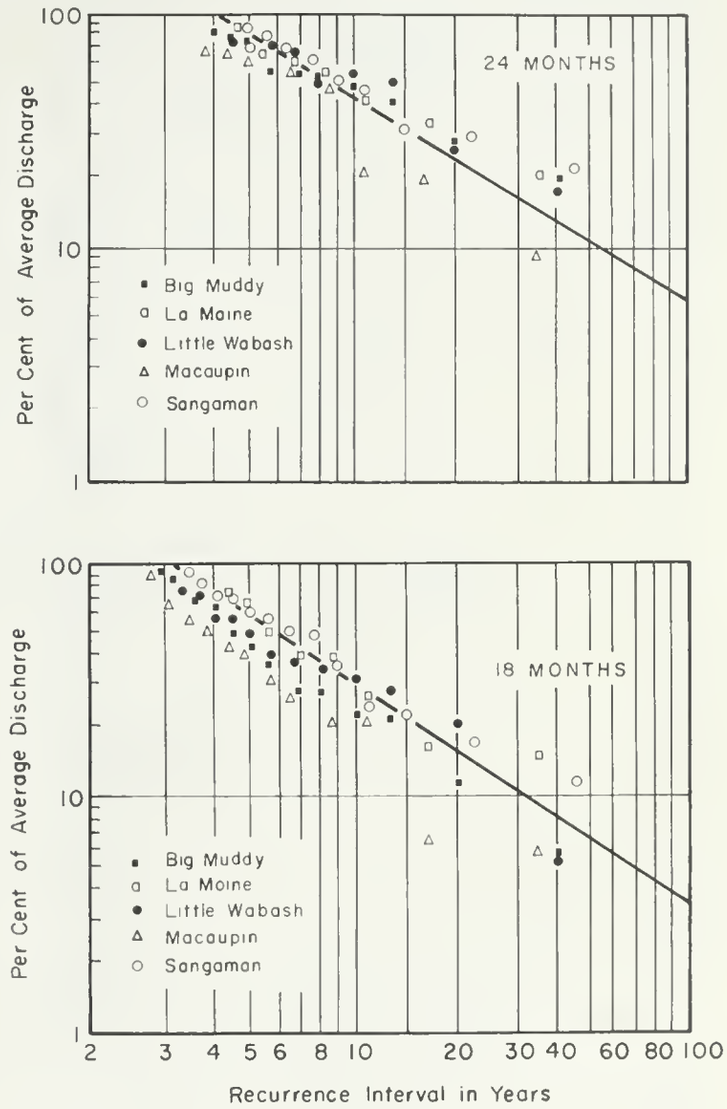


FIGURE 26. LOW-FLOW FREQUENCY DATA, CONSOLIDATED ON BASIS OF PER CENT OF AVERAGE FLOW.
18- and 24-Month Duration.

This topic deserves further study, but for the present (since the concordance of the data appears good) generalized estimates prepared on the basis of curves such as those shown in Figure 25 appear to produce serviceable values.

Because of possible significant differences in variability of flows from place to place, the caution is repeated that studies at any particular location should be founded on data from the immediate vicinity of the proposed development.

In order to verify further the consistency of the data obtained, a graph of the relationship between runoff (expressed in per cent of the annual streamflow) and the duration of the low-flow period was prepared, with a separate line for each of the following recurrence interval values: 4, 10, 20, 40 and 100 years. On this graph (Figure 29), using values from the lines sketched through the consolidated data in Figures 25 to 28, smooth curves through the various values were possible. In the initial preparation of the work drawing for Figure 29, some discrepancies in plotting positions for the long-duration low-flow period (30 to 60 months) were noted. By drawing smooth curves on Figure 29, and adjusting the lines shown in Figures 25 to 28, consistent results, in good accord with the basic data, were obtained in the placing of the lines in Figures 25 to 28.

When the data for the five stream-gaging stations were consolidated in Figure 29 the tendency toward appearance of separate families of curves seemed to be lost. Seasonal variances might have been expected to cause the curves for various recurrence intervals in Figure 29 to be undulatory in accordance with the annual cycle. There was some indication of such behavior, but it is thought to be partially obscured by the fact that the duration periods chosen for study were in multiples of half-years. It is believed that study of low-flow duration periods of all lengths that are multiples of a quarter of a year or of one month, would reveal the effect of seasonal variations. Furthermore it is reasonable to expect that low-flow periods having durations slightly shorter than integral multiples of years, might indicate more adverse values than are shown in Figure 29. This would follow from the observation that the majority of periods of extreme low flow tend to commence in late spring. It appears that study along these lines is desirable, and might be expected to indicate departures from the results brought out by the present study.

Figure 29 includes an asymptote, which should be approached by the curves characterizing the data.

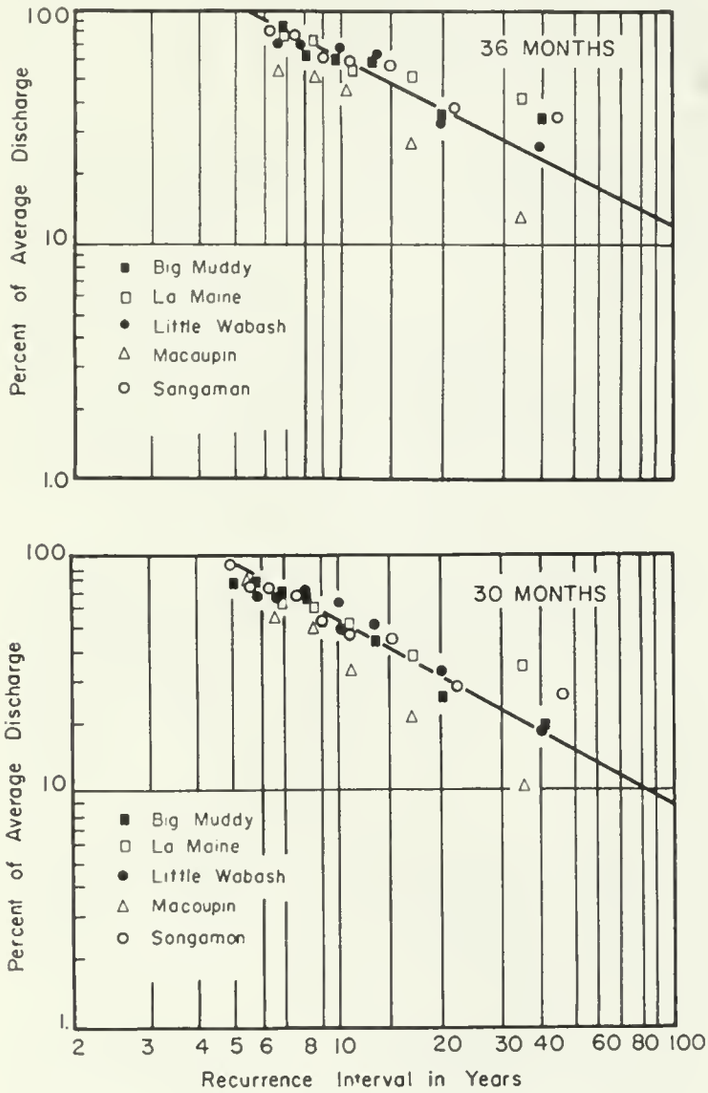


FIGURE 27. LOW-FLOW FREQUENCY DATA, CONSOLIDATED ON BASIS OF PER CENT OF AVERAGE FLOW.
30- and 36-Month Duration.

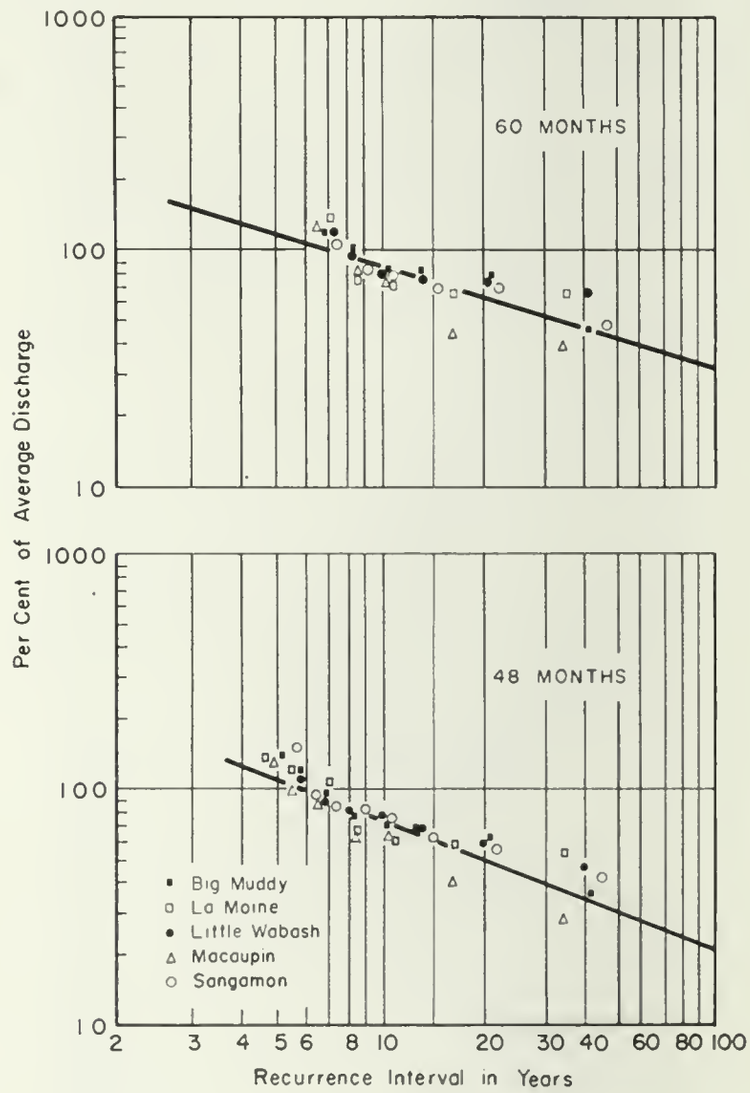


FIGURE 28. LOW-FLOW FREQUENCY DATA, CONSOLIDATED ON BASIS OF PER CENT OF AVERAGE FLOW.
48- and 60-Month Duration.

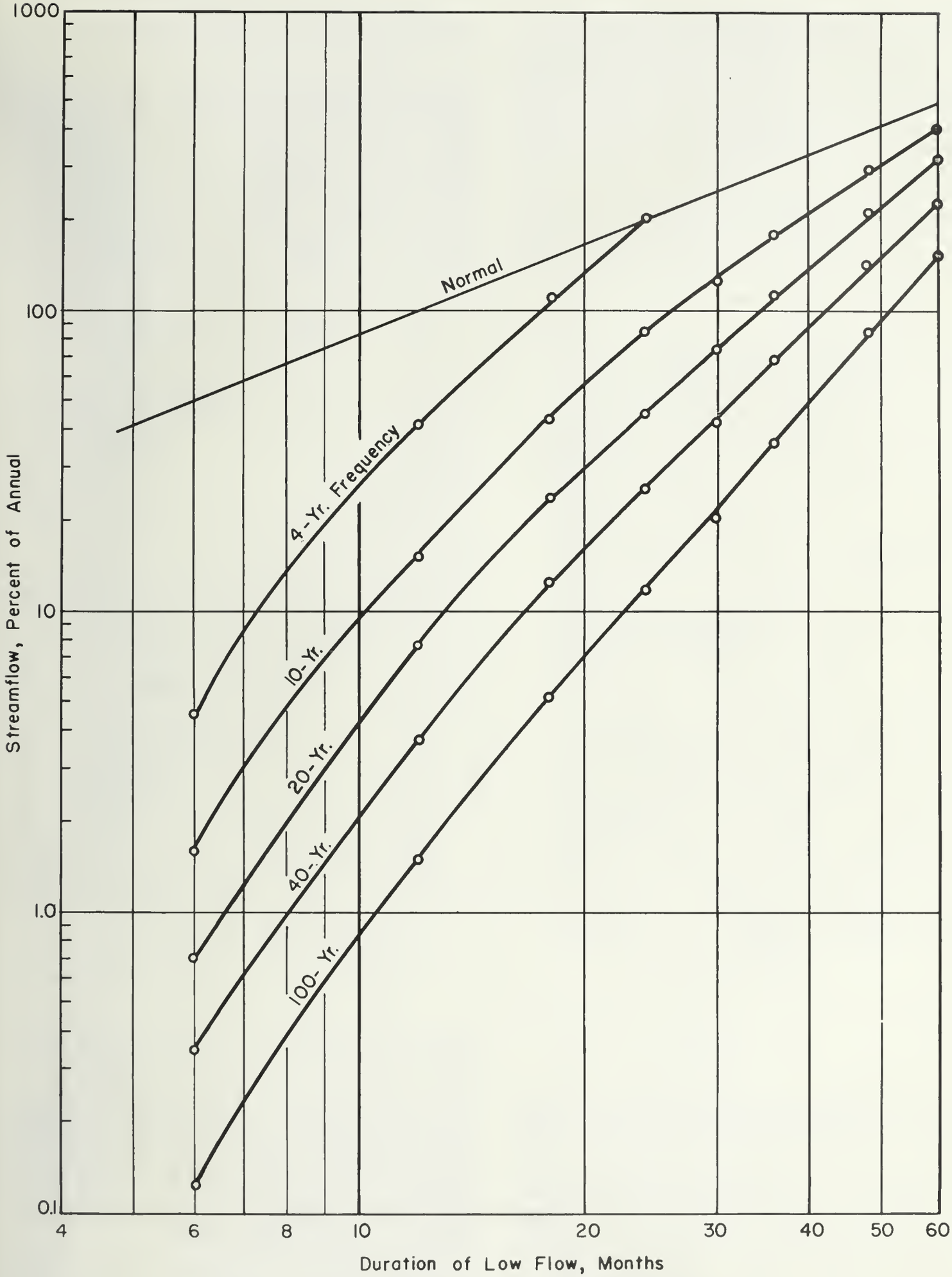


FIGURE 29. GENERALIZED RELATION BETWEEN DURATION AND FREQUENCY OF PERIODS OF LOW STREAMFLOW, SOUTHERN ILLINOIS.

This curve was constructed on the assumption that six months' flow would be 50 per cent of the annual average, that two years' flow would be 200 per cent of the average, etc. Some consideration was given to the points at which the curves for various recurrence intervals should approach the asymptote. It would appear that, in Figure 29, the 4-year recurrence interval curve should approach or join the asymptotic line at a value having an order of magnitude of approximately two years. The data bear this out. It would further appear that a 10-year recurrence interval curve should approach or intersect the asymptote at a value having an order of magnitude of approximately five years' duration. The data are in fair agreement with this concept.

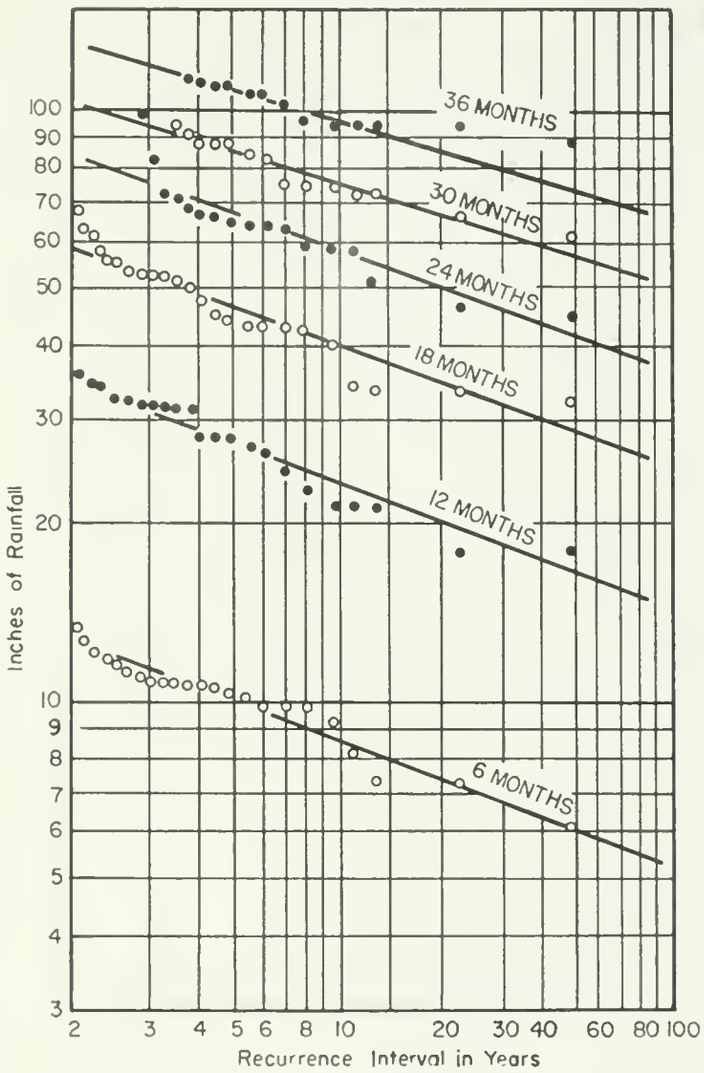


FIGURE 30. FREQUENCY OF PERIODS OF LOW PRECIPITATION, ST. LOUIS, MISSOURI.

It appeared that it might have been desirable to calculate streamflow values in per cent of median flow rather than in per cent of mean flow in order to enable more complete consideration of the data in Figure 29. However, analysis of the data showed the mean and median flows to agree quite well with the exception of those for six months duration, so the computations based on averages were retained.

In this report the approach used in working up the data on probability of low-flow periods of various durations has been empirical. Theoretical bases are available which indicate that relationships between frequencies for the periods of various lengths can be determined mathematically rather than empirically. A method for this has been suggested by Alexander[24] which makes use of the coefficient of variation for each stream, together with the mean flow for the stream. That there may be these differences is suggested by recent work of Mitchell[25] on floods in Illinois.

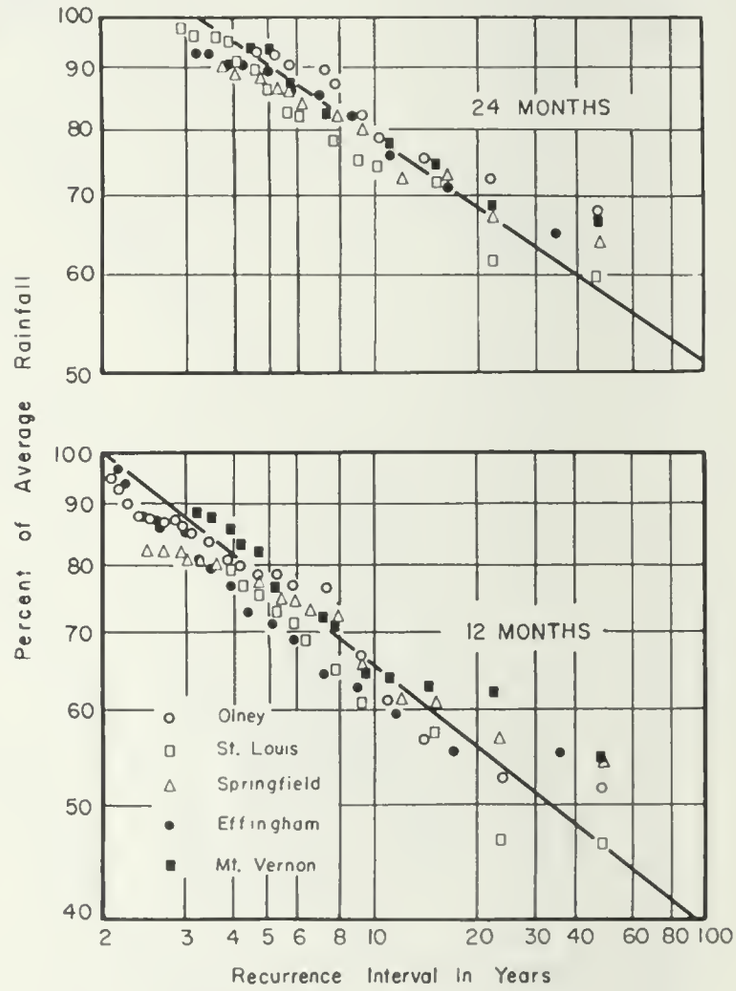


FIGURE 31. LOW RAINFALL FREQUENCY DATA CONSOLIDATED ON BASIS OF PER CENT OF NORMAL.

The mathematical approach might make use of conventional probability methods for determination of the recurrence interval (reciprocal probability) at which curves, such as those in Figure 25, intersect the value of mean flow. The slopes of the lines should be related to the coefficient of variation of the stream under study.

The problem is complicated by the persistent seasonal cyclic variance of both streamflow and rainfall so that simple solutions may possibly be

found only for integral multiples of years. Others have dealt with this[17], [26] by separating the statistical approach into two components: annual storage and monthly storage. The former takes care of year-to-year variations and the latter is intended to take into account the seasonal variations. The desirability of this separation, may be questioned and there are reasons to believe that it will be necessary to consider annual and monthly storage together in some fashion, for they seem to be related. In this study, for example, extraordinary seasonal deviations from normal were observed during 1952-55. These seem to have been the result of the unusually low annual flows, rather than independent phenomena.

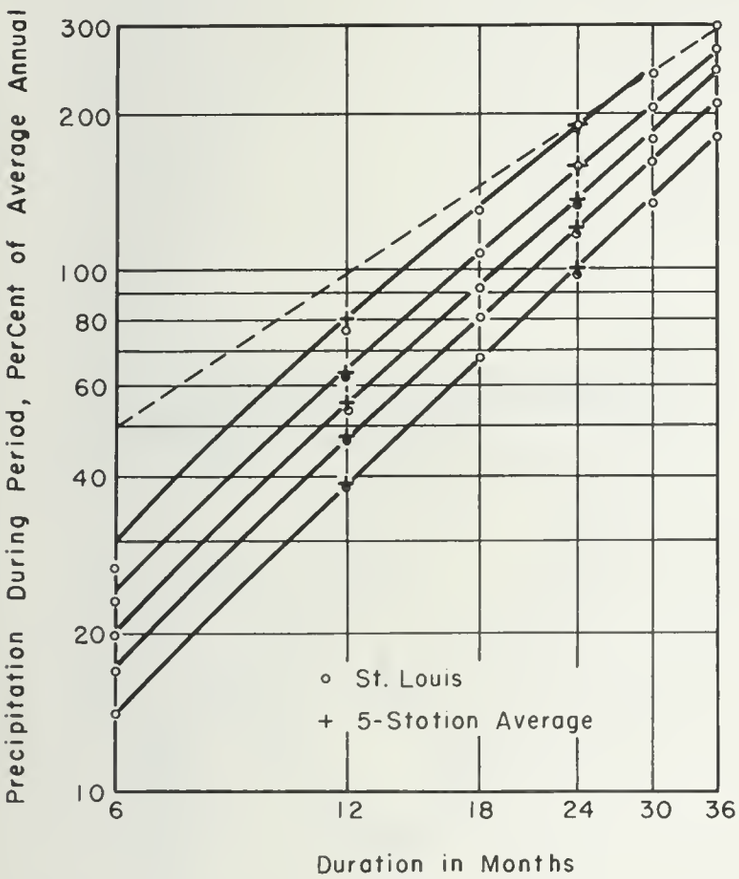


FIGURE 32. GENERALIZED RELATION BETWEEN DURATION AND FREQUENCY OF PERIODS OF LOW RAINFALL (SOUTHERN ILLINOIS).

To return to the possible application of a mathematical approach, Alexander's presentation indicates [24] that there should be a theoretical fixed relation between the probability of occurrence of a one-year drought and those for droughts of other lengths. Assume for example a location for which the recurrence interval is 50 years (probability 0.02) for streamflow of one inch or less in 12 months. Flows lower than one inch could be considered "failures" and higher ones "successes." It would appear possible to calculate the probability of occurrence of flows equal to or smaller than one inch in 2, 3, or "n" successive years from Alexander's presentation.

Frequency of Periods of Low Rainfall. A method similar to that used for streamflow data was used for unifying the data from the five rain-gaging stations, in which the per cent of normal rainfall was plotted against recurrence interval. Again the result was better than when absolute rainfall amounts were used. The data are shown in Figures 30 and 31.

As has been mentioned above, values of totals for 12- and 24-month duration periods were computed for the five rain-gaging stations, but 6, 18, 30, and 36 month totals were computed only for St.

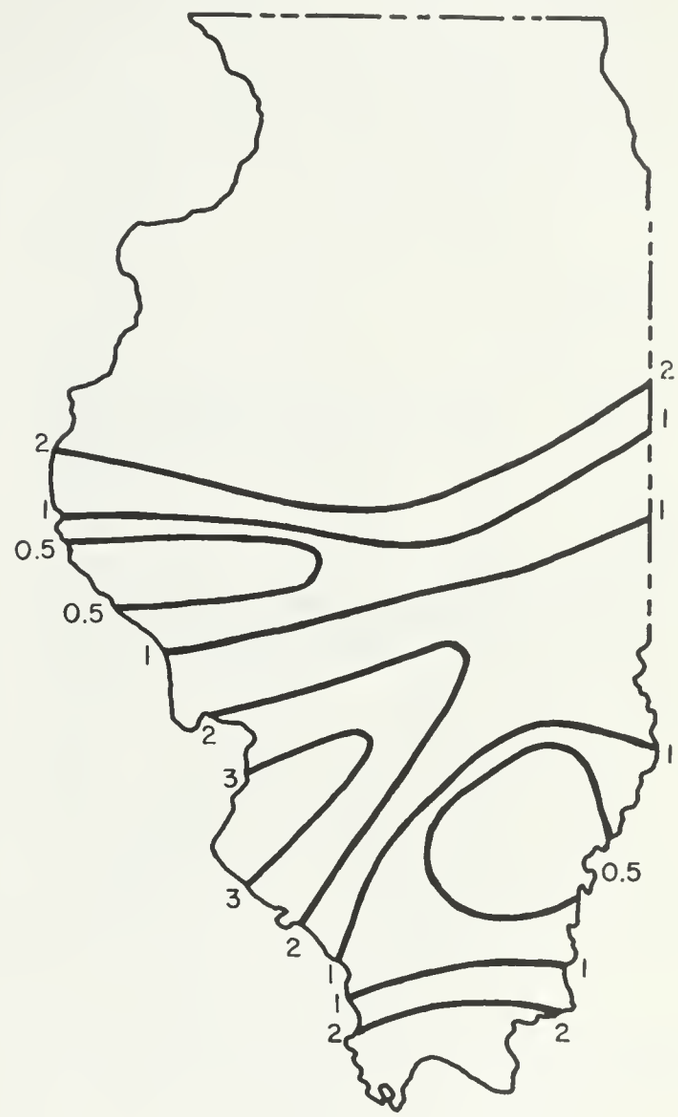


FIGURE 33. PER CENT OF NORMAL RUNOFF, 6 Months Ending January 1954.

Louis. Average annual rainfall at St. Louis is 37.02 inches. These data have been brought together in Figure 32 (which is similar in construction to Figure 29) in which representative consolidated curves for the five-station rainfall average have been constructed for all periods from six months to 36 months.

The difference in slopes of curves noted for runoff do not seem present in the rainfall frequency consolidated curves.

It should be remembered that the data for the rain-gaging stations are point observations, while those for the stream-gaging stations are measurements that integrate the occurrence of runoff from the areas gaged.

Recurrence Interval of 1952-55 Drought

The data available for the 1952-55 drought enable estimation of the severity of the drought on a basis of rainfall and streamflow. The maps in Figures 13 to 18 show the distribution of runoff during extreme low periods. Since better correlation of the frequency data was obtained using "per cent of normal" as the basis of comparison than by using absolute runoff data, runoff maps for the extreme low periods were prepared showing these values in per cent of normal. These maps were constructed by an overlay procedure using Figure 6 and Figures 13-18 as the bases. The maps are presented as Figures 33 to 38 and no allowance was made for seasonal variations.

The areas within contours and within the Illinois boundaries were planimetered. For purposes of analysis these data were plotted to produce runoff distribution curves from which values of runoff could be taken for areas of various sizes. (Figure 39.)

The consolidated streamflow frequency curves (Figures 25-28) are for drainage basins whose areas range from 550 to 1310 square miles, with a median value of 930 square miles. The frequency curves for the rainfall data (Figure 31) are for point measurements. Accordingly, to estimate the return frequency of the 1952-55 drought, it is necessary to use two different bases for entering the frequency curves. For the runoff frequency estimation, the value of runoff for a drainage area of 1000 square miles was taken off the curves in Figure 39. These, and the recurrence interval values for the 1000 square mile areas of lowest flow are given in Table 10.

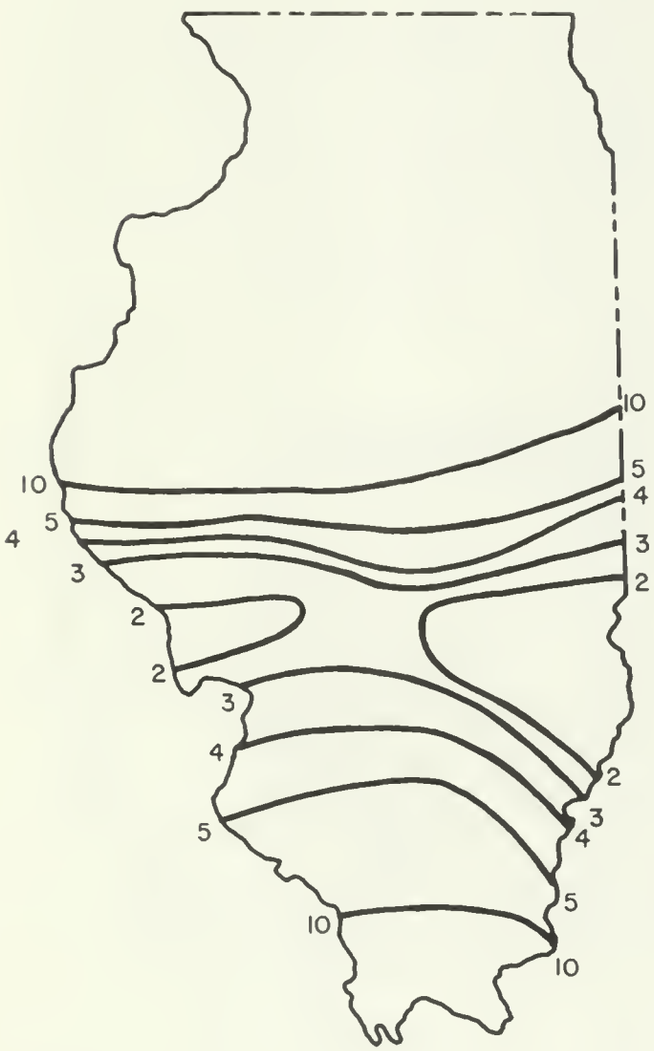


FIGURE 34. PER CENT OF NORMAL RUNOFF, 12 Months Ending December 1954.

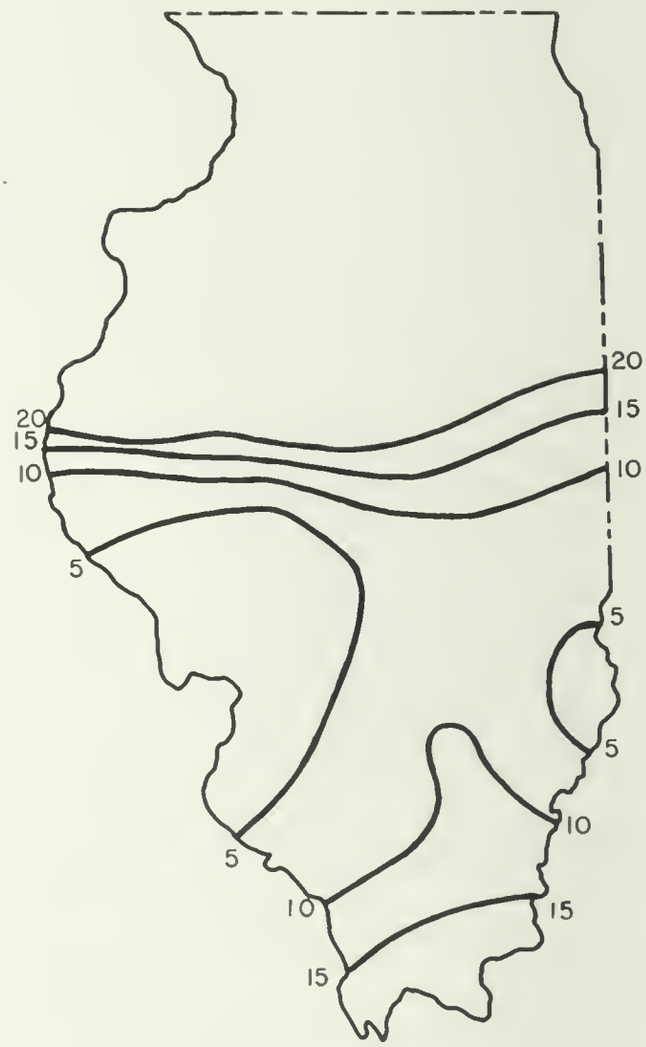


FIGURE 35. PER CENT OF NORMAL RUNOFF, 18 Months Ending December 1954.

Attention is called to the fact that 30-month streamflow was less than that for 24 months, when both were expressed as per cent of normal. This was not true of the actual amounts in inches. This reversal of order is believed due to seasonal effects which are discussed elsewhere in this report. Note that the 36-month period chosen was for the period ending April 1955. The 36-month totals rose in May, declined in June and rose again in July 1955, all hovering close to the April minimum. It was therefore not certain that the 3-year period ending with May 1955 was the lowest during the recent drought, but complete data for August were not available at this writing.

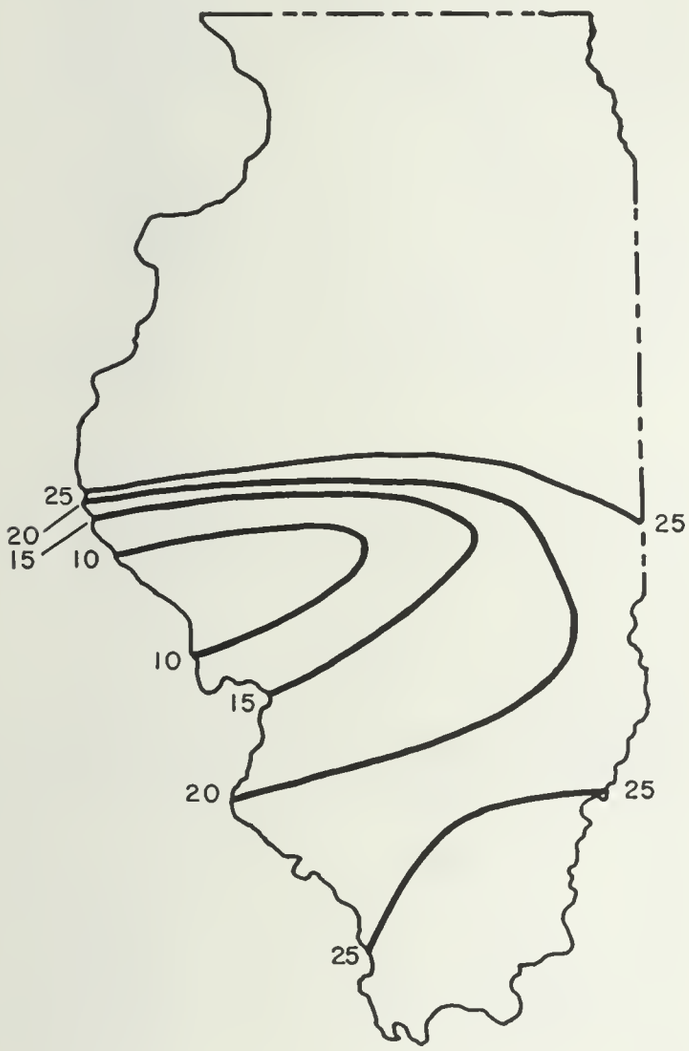


FIGURE 36. PER CENT OF NORMAL RUNOFF, 24 Months Ending December 1954.

For estimation of recurrence interval of the rainfall data, lowest point values for stations in the vicinity of the area of lowest streamflow were determined. In working these out, sliding totals for the individual rain-gaging stations in the area were prepared for 12- and 24-month periods. The lowest 12-month rainfall value found was for Grafton, 16.48 inches which gave 43.7 per cent of normal for the period ending July 1954. The lowest 24-month rainfall value found was for the period ending July 1954 at Grafton, where the rainfall for the two-year period was 39.80 inches or 52.7 per cent of normal. Using these rainfall values, and entering the curves in Figure 31, recurrence intervals for the 12- and 24-month periods of low rainfall were found to be 63 and 87 years respectively.

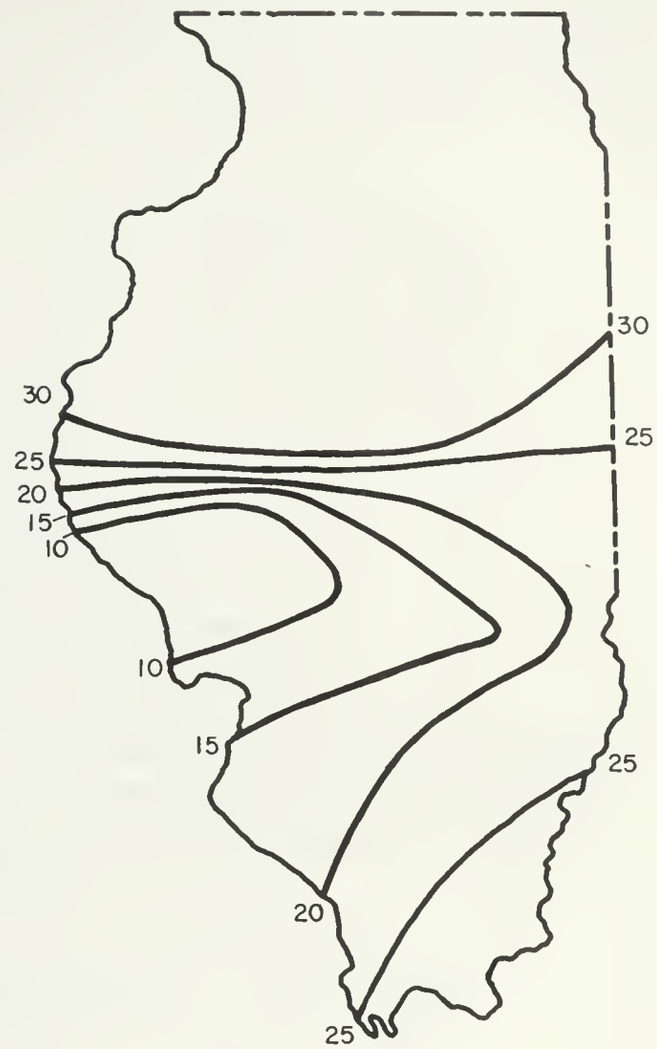


FIGURE 37. PER CENT OF NORMAL RUNOFF, 30 Months Ending December 1954.

Earlier trials of the determination of point rainfall values by the area-distribution graph method used for runoff had indicated that this method yielded recurrence interval values approximately half those obtained when the lowest point observation in the area of low rainfall was used. The use of the lowest point rainfall values obtained gave recurrence interval values that checked closely with those obtained for streamflow by the area-distribution graph method for 1000 square miles.

On the basis of point rainfall data for the period 1906-54 and streamflow data for 1914-54,

the drought of 1952-55 appears to have been of a severity that would occur on the average of once in 60 to 100 years. On the basis of streamflow, the 3-year period ending June 1955 appears to have been a more unusual event than the worst periods of a two year or less duration occurring during the period. Estimates of recurrence interval based on streamflow ranged from 75 to more than 100 years.

Median value of the estimates of frequency of the 1952-55 drought gave a recurrence interval of 83 years.

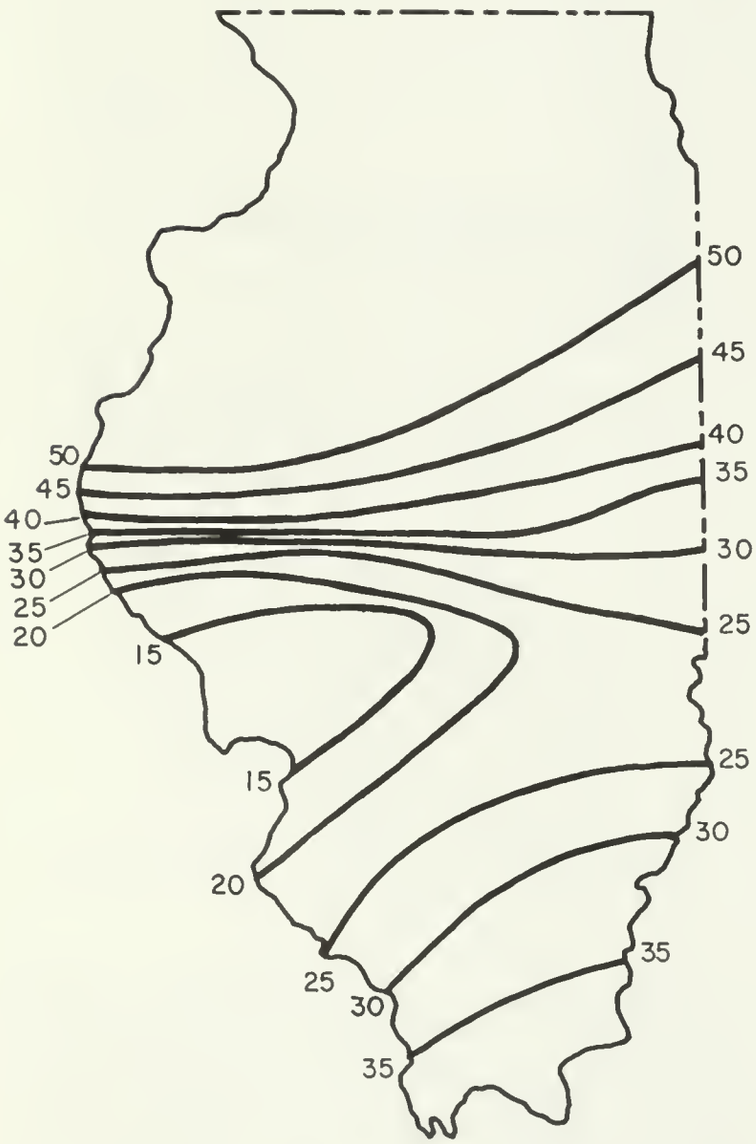


FIGURE 38. PER CENT OF NORMAL RUNOFF, 36 Months Ending April 1955.

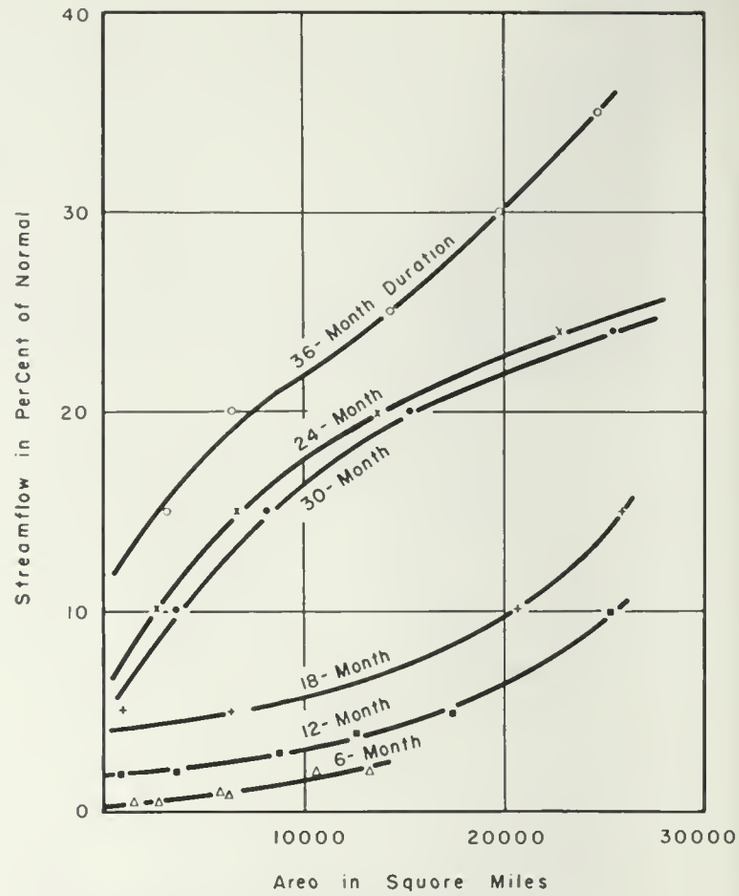


FIGURE 39. AREAL DISTRIBUTION OF RUNOFF IN DROUGHT ZONE IN ILLINOIS. 1952-1955.

TABLE 10

RECURRENCE INTERVALS OF PERIODS OF LOWEST STREAMFLOW OF VARIOUS DURATIONS DURING 1952-1955

SOUTHERN ILLINOIS

Duration in Months	Streamflow in per cent of normal	Recurrence Interval in Years
6	0.35	75
12	1.9	83
18	4.2	82
24	7.4	77
30	6.1	100+
36	12.6	100+

ESTIMATING IMPOUNDING-RESERVOIR STORAGE

INTRODUCTION

Earlier in this report it was brought out that the greatest incidence of shortages of water occurred in communities relying on impounding reservoir sources and that, while unexpected increases in demand for water accounted for more shortages than any other cause, many of the difficulties were attributable to the inadequacy of impounding reservoirs. These findings suggest the desirability of a review of existing practice in design of impoundments in relation to the low-flow frequency data in the preceding section. The low-flow frequency and duration data presented furnish a basis for design of future impoundments.

For municipal or industrial water supply, the controlling considerations in deciding on reservoir capacity are adequacy of supply and cost. While the economics of determining reservoir size is not within the scope of this report, economic considerations confront the designer at a number of points.

The engineer, in evaluating reservoir sources of supply, examines the possible available sites in the vicinity that he believes might be practicable in relation to estimated demand. These will include those locations at which dams (with appurtenances such as spillways) could be built at reasonable cost. This consideration usually requires a narrow section of valley such that the length of dam is not excessive. It also favors the choice of the smaller drainage areas above the dam-site in order to minimize the size of spillway. The engineer will also prefer deep, steep-walled valleys, since they minimize evaporation losses and land-acquisition costs. Deep reservoirs also yield cooler water than shallow ones. Sites at higher elevations will be preferred to those of lower ones in order to minimize pumping costs. Sites closer to the point of use will be preferred to those more remote, in order to minimize length of conduit from the source to the point of use. The influence of distance from point of use to source has in recent years been subject to some noteworthy exceptions in which existing natural waterways were used to convey the water from the source to the point of use. Thus, the cost of constructing a pipeline is saved. However, the use of natural waterways is accompanied by loss of water due to seepage through the channel bottom, through evaporation, and use by riparian vegetation. These losses may range from negligible values to more than 50 per cent of the total [27] [28], depending on local conditions.

Economic considerations for various sites being equal or nearly equal, the engineer then evaluates more precisely the adequacy of each available source. He estimates the amount of water that will be required per unit of time, and establishes a minimum drainage area.

Review of Methods for Calculating Storage

It is desirable to build a reservoir on a watershed of the maximum feasible area in order to obtain adequate runoff, and on the smallest feasible area to minimize silt damage. It is therefore of critical importance to determine a suitable drainage area for which an economical reservoir site is available.

The early surface water reservoirs built for public water supply in Illinois are reported to have been designed on the basis of New England runoff data. This practice probably developed out of the classic studies of Allen Hazen[17] which were published in 1914. Hazen analyzed the desirability of storage provision for water supply from the point of view of low-flow frequency, using data primarily from the New England states. Additional information from other parts of the country was brought together by Hazen in a subsequent publication[23]. The methods used by Hazen, while comprehending nearly all the important factors, are complex, and do not seem to have been adapted for use in Illinois design practice.

Instead of working out suitable applications of Hazen's method, designers in Illinois seem to have followed the practice of using rainfall data and applying runoff factors to these for estimation of streamflow. The files of the Water Survey contain a number of engineering reports on reservoir design prepared prior to 1930 for which it was assumed that minimum runoff would be of the order of 10 per cent of the annual rainfall.

As a greater number of Illinois stream-gaging records became available, data from extreme low-flow periods began to be used for design. Synthetic mass curves were sometimes prepared for design purposes, by transposition and adjustment of these records. This latter procedure remained popular until 1950, when W. D. Mitchell made an extensive analysis of streamflow records in Illinois through 1945 by means of residual mass curves, and described improved techniques for determining storage requirements for various draft rates[7].

It might seem possible to obtain a yield from an impounding reservoir equal to the mean runoff from the watershed. In actual practice, since yields must be attained by use of reservoir construction, losses due to seepage and evaporation occur, and these range from a small fraction of the use to an amount exceeding the use, depend-

ing upon reservoir surface area and underground conditions at the reservoir.

In addition, much larger reservoirs would be required for use of a high proportion of the streamflow than have generally been built. Mitchell's data for the Little Wabash River at Wilcox for the period 1939 to 1942, indicates that there would be no spillage for a period of 21 months if a reservoir were built to utilize 40 per cent of the average discharge, and that this period would extend to 36 months if it were desired to use 60 per cent of the mean streamflow. These times make no allowance for evaporative or seepage losses, except to the extent that such losses have been included in the draft rates. In Midwest engineering practice, consideration has seldom been given to drawdown periods exceeding three years, although in other parts of the world longer periods of drawdown are expected[29] [30]

The storage requirements indicated by Mitchell's study vary with the demand (draft rate) on the reservoir. To determine this relationship for the southern half of Illinois, averages of the unit storage requirements which he worked out for eight drainage areas have been calculated. These were for the Sangamon River at Monticello, South Fork of the Sangamon River at Kincaid, LaMoine River at Ripley, Macoupin Creek at Kane, Big Muddy River at Plumfield, Embarrass River at Ste.Marie, Little Wabash River at Wilcox, and Skillet Fork at Wayne City. From Mitchell's draft-storage curves for these basins, the storage ratio in million gallons per square mile was determined for each basin for draft ratios (Draft ratio is ratio of demand to average discharge, and may conveniently be expressed in per cent) of 30, 40, 50 and 60 per cent. The means of these data were determined and are represented by the curve in Figure 40.

The data given in Figure 40 are based on the worst conditions that occurred during the periods of record on the eight basins included in the study. The curve therefore represents requirements for an indeterminate return frequency, probably in excess of 20 years, since the records involved in the study ranged from 16 to 34 years in length. The curve gives no information on low-flow duration, but most of the values from which it was determined were for two- to three-year low-flow periods, especially at the higher draft rates. Richard Hazen has pointed out[31] that it would be desirable to construct such curves on a basis of low-flow frequency, but hitherto data in the Midwest have been insufficient to permit this. Inasmuch as Figure 40 does not include consideration of the 1952-55 drought, it should be considered only as a general guide, and storage requirements exceeding those indicated in the curve may occur. The curve does illustrate the necessity for sharply increasing storage capacity as the draft rate is increased.

A simplified version of the Allen Hazen approach to storage computations, published recently[32], explains the allowance for siltation losses, which was not treated in the earlier publication by Hazen. The handling of runoff data was the same as that of Hazen, in that annual deficiencies were computed, and these were subsequently arrayed to determine the frequency of

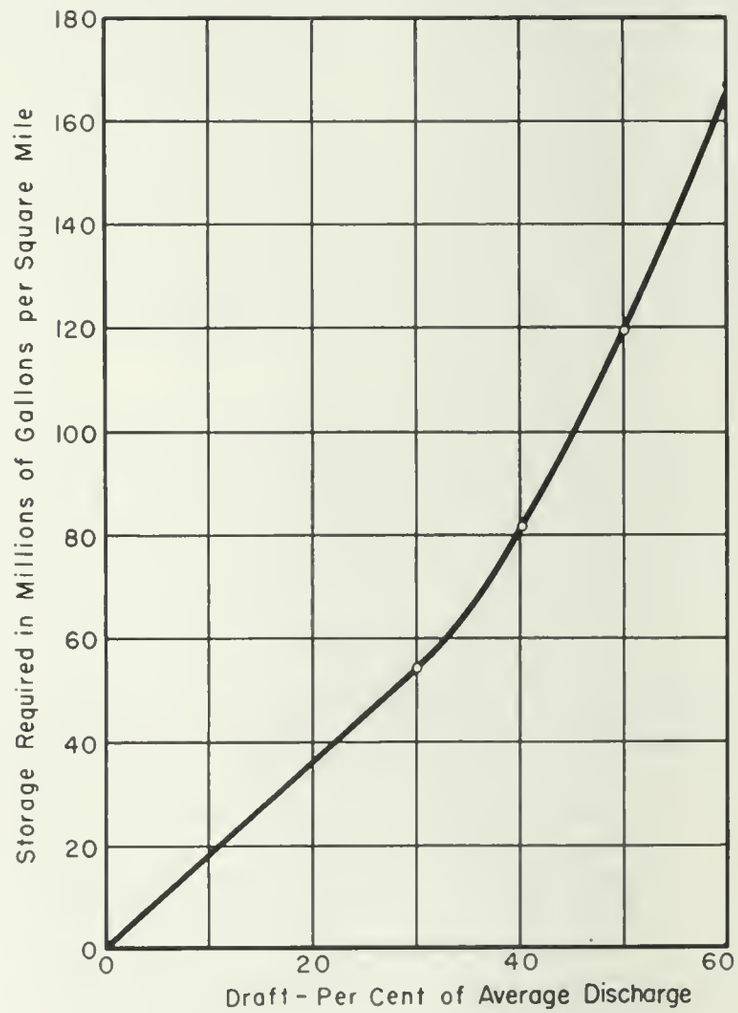


FIGURE 40. DRAFT-STORAGE CURVE FOR SOUTHERN ILLINOIS BASED ON AVERAGE OF 8 GAGING STATIONS.

occurrence of annual storage requirements. Smallwood and his colleagues improved on the Hazen approach by using the mass-curve technique, which gives values extending beyond the end of the water year or calendar year. Hazen's method of determination was arithmetic rather than graphic and started from the same month each year in making computations. There is question whether these approaches obtain "independent" values. It is con-

ceivable that several values from within one particular low-flow period might be obtained, particularly under the arithmetic method used by Hazen. The incidence of dependent values could have a profound effect on frequency determinations.

Suggested Design Criteria

In the preceding section, the available procedures and methods for estimating capacity of impounding reservoirs were discussed. It was pointed out that the method most readily available made use of the most extreme periods on record, and computed the storage requirements for these periods[7]. This procedure does not tell us the natural risk which the reservoir is prepared to meet; it merely states that the reservoir can meet a situation as adverse as was experienced during a certain recorded period. It would obviously be desirable to know the probability of occurrence of the "design drought." An analysis of storage requirements that includes the frequency of occurrence of low flows of various durations as a consideration has therefore been made.

This analysis makes use of the unified data on low-flow duration and frequency described in a preceding section, which led to development of a method similar in some respects to that originally used by Hazen[17] in which storage volume requirements, expressed in per cent of average annual discharge, are related to draft rate in per cent of average flow rate.

Stated in general terms, the method developed involves calculation, for fixed low-flow frequencies, of the quantities of water that will be withdrawn from storage at various draft rates for various duration periods. For each given low-flow frequency and draft rate, one value of duration will prove to cause the maximum withdrawal from storage. These maxima may then be used to establish a separate graphic relation between storage required and draft rate for each low-flow frequency. The critical values of duration may also be used to establish the relation between draft rate, storage provided, low-flow frequency, and duration of recession in the reservoir.

For calculating storage required, confusion is avoided by expressing the data in units of streamflow. From Table 4 it was determined that the mean discharge in the region studied was 10.0 inches per year. Draft rates were then expressed in inches, with five inches per year corresponding to a 50 per cent draft rate, etc. A draft rate of five inches per year for a two year period would constitute a total draft of 10 inches. From the total draft was subtracted the value of runoff for the corresponding duration period, obtained from Figure 29 (converted to inches), to yield the volume of flow in inches that must be provided from the reservoir for a given low-flow frequency. Multiplying these values by 10 gave the volume of storage in per cent of mean annual discharge. These data on quantity of storage required for each low-flow duration period, draft rate and low-flow frequency were then plotted in residual mass diagram form, as illustrated in Figure 41, a separate drawing being prepared for each recurrence interval.

The shapes of the curves in Figure 41 and in those for other recurrence intervals were quite similar, although maxima were not always attained within the 60-month period studied. It appears that, in order to work the data out with full precision, it would be desirable to obtain data for low-flow periods longer than 5 years. The curves

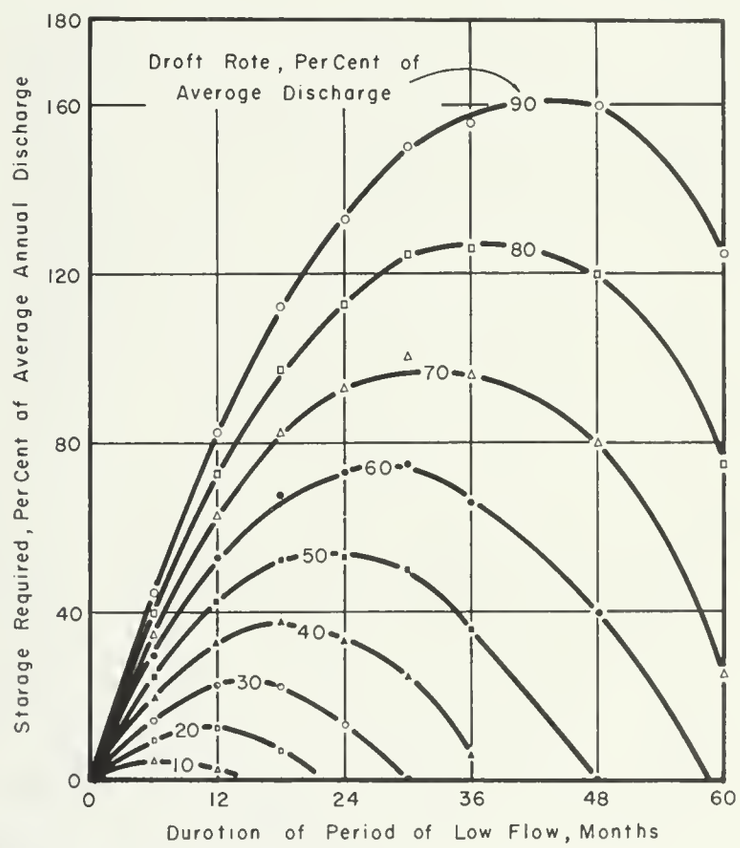


FIGURE 41. RESIDUAL MASS DIAGRAM OF STORAGE REQUIRED FOR PERIODS OF LOW FLOW OCCURRING ONCE IN 20 YEARS.

in Figure 41 show that for a given draft rate the amount of water withdrawn from storage increased to a maximum as the duration of the low flow increased. As duration further increased storage required subsequently declined. It is evident from Figure 41 that, for a 20-year recurrence interval, the duration of the period during which total streamflow is less than 10 per cent of the mean flow is about six months but that, for the same recurrence interval, the duration of the period in which the total flow is less than 90 per cent of the mean flow is approximately 44 months. Increases in streamflow after these duration times account for the subsequent decline of the residual-mass curves.

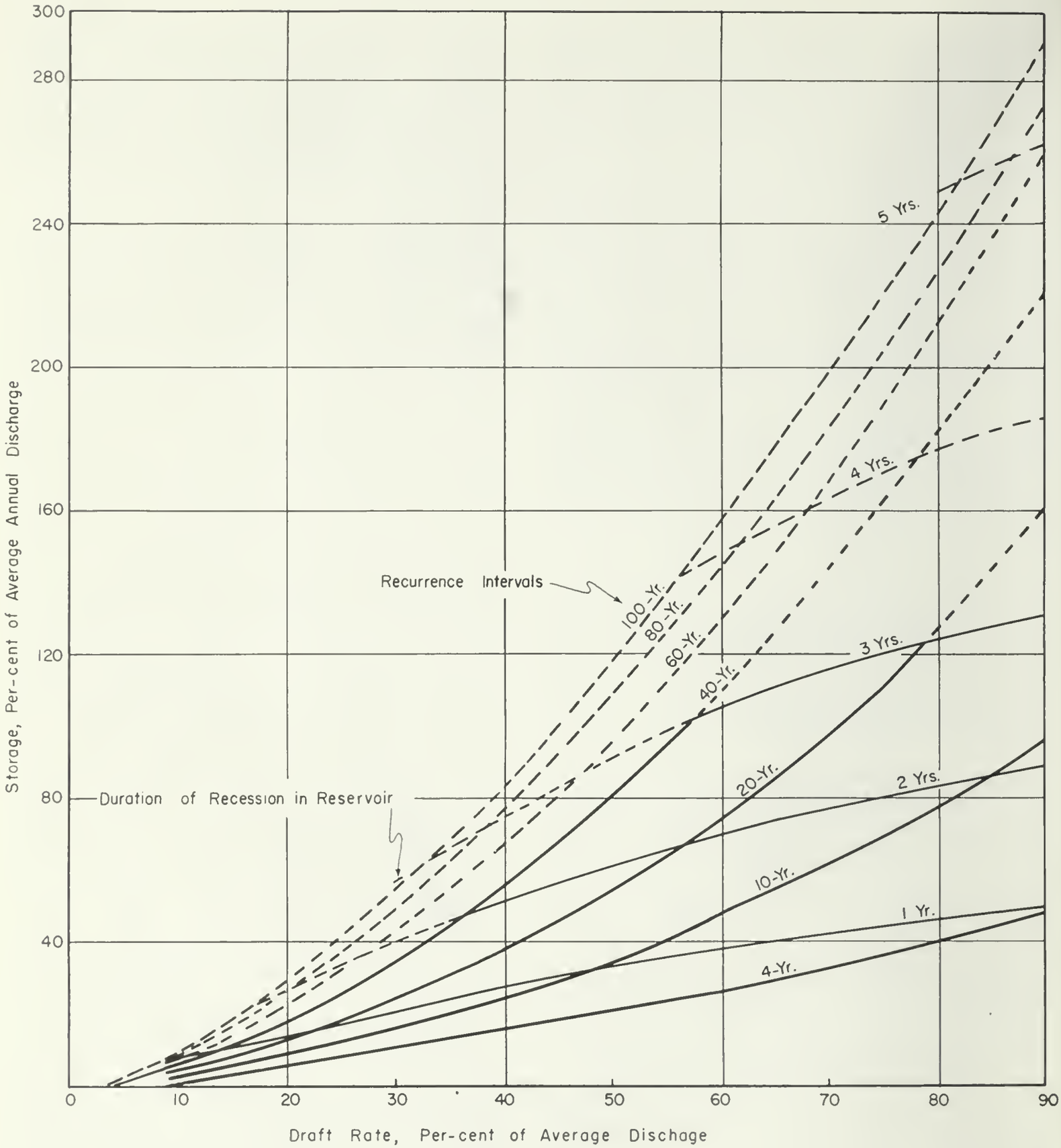


FIGURE 42. STORAGE REQUIRED FOR MAINTENANCE OF VARIOUS DRAFT RATES DURING LOW-FLOW PERIODS OF VARIOUS RECURRENCE INTERVALS AND DURATIONS.

In Figure 41, the location of the apex of each curve determines the amount and time of maximum drawdown in the reservoir. Values for larger periods of duration represent the recovery phase. Computation of these curves to duration periods exceeding 5 years in length could enable determination of the total time of drawdown. The maximal values from Figure 41 and from similar graphs for other recurrence intervals were consolidated in Figure 42 which gives generalized values, for any draft rate of 90 per cent or less and recurrence interval of 100 years or less, of the amount of storage necessary and the duration of the critical low-flow period in the southern half of Illinois.

In Figure 42, those portions of the graph that are based on interpolated values from the low-flow frequency curves are shown as solid lines, while those parts that are extrapolated beyond the period of record are shown as dashed lines. The heavier lines in Figure 42 show the quantity of storage required to sustain various draft rates, while the lighter lines show the duration of the low-flow period that would produce the maximum drawdown in a reservoir of stated capacity at given rates and recurrence intervals.

To illustrate the use of Figure 42, consider the case in which it is desired, at a given site, to develop 60 per cent of the average discharge with assurance that the supply will not fail, on the average, more often than once in 20 years. Figure 42 shows that the necessary volume of storage would be equal to 74 per cent of the mean annual discharge, and that the period during which water levels would decline in such a reservoir under the 20-year low-flow condition would be about 26 months. If, on the other hand, it is desired to develop 60 per cent of the mean discharge of the stream in such a way that the supply would be exhausted only once in 100 years, the volume of storage required would be 158 per cent of the mean annual discharge, and the duration of recession of reservoir levels would be approximately 50 months.

To the storage requirement estimates made in this fashion must be added adjustments for loss in storage due to future sedimentation. Allowances for losses due to evaporation would be subtracted from draft rates to determine net yields.

Figure 42 may also be used to evaluate existing reservoirs. In computing draft rate, the effect of evaporation should be added to the withdrawal. The total, divided by the mean discharge at the site (which may be estimated by use of Figure 6) equals the draft rate. This figure may be used as one value for entering Figure 42. The total volume of storage in the reservoir, divided by the mean annual discharge, gives the storage figure, which is the other value needed for entering the graph. From the point on the graph located by these two values, the capability of the reservoir in meeting demands may be interpolated in terms of low-flow frequency which the reservoir can safely meet. The duration of the recession may also be determined in the same fashion. Assume, for example, a reservoir which is capable of storing one year's normal streamflow. Assume that the draft rate from the reservoir is 50 per cent of the average annual flow. From Figure 42 it may be seen that this reservoir would be capable of maintaining the assumed draft rate for the kind of low-flow period that occurs once in 67 years, and that such a low-flow period could have a duration of approximately 3-1/2 years.

Figure 42 may also be used to evaluate the usefulness of water conservation measures. For example, assume a situation in which the draft rate is 60 per cent and the storage volume is 100 per cent of the mean annual discharge. Such a reservoir-watershed combination would be estimated from Figure 42 to be capable of meeting a low-flow period of a magnitude that would occur once in 35 years. If means could be found to reduce the draft rate to 50 per cent, the reservoir could be made capable of meeting a low-flow period having a 67-year recurrence interval. To attain a substantial reduction in draft rate requires a considerable reduction in pumpage, for the demands of evaporation continue, unreduced. Inasmuch as the desirability of such a conservation measure could become apparent at the end of 18 months of drawdown, which is approximately half of the anticipated total period of recession, a reduction in pumpage of approximately double the amount estimated would be necessary to achieve the average reduction required. In many communities, such reduction is possible through eliminating leakage, unaccounted-for water, limiting free public use, and through applying restrictions. It therefore appears possible, under certain circumstances, to substantially extend the usefulness of a reservoir through conversation measures. Chief among these measures should be the initiation of continuing leak detection and elimination programs.

Reservoir Sedimentation

Reservoir sedimentation is a factor that was not taken into account in the design of many of the early impounding reservoir supplies in Illinois. This was due to complete lack of data on sedimentation. As a result, many of the older reservoirs in the state have lost storage capacity at surprisingly high rates. These losses of capacity have frequently deprived municipalities of necessary storage space, and thus contributed to supply shortages.

In estimating the watershed area desirable for a reservoir project, the engineer must have two opposing considerations in mind: (1) obtaining sufficient runoff to serve the community's needs, and (2) minimizing the drainage area so as to secure the least sediment damage to the reservoir. While the sediment that reaches a reservoir increases as the drainage area is increased, as shown in other studies [33] this relationship is not one of direct proportion.

For estimating sediment damage rates, it is convenient to use the capacity-watershed (C/W) ratio, which is expressed in acre-feet per square mile. A small C/W ratio usually indicates little space for storage of sediment in respect to sediment yield, and a large ratio indicates great space for storage. Accordingly, reservoirs with low C/W ratios accumulate sediment more rapidly (percentagewise) than those with high ratios. To a small extent this is offset by the effect of

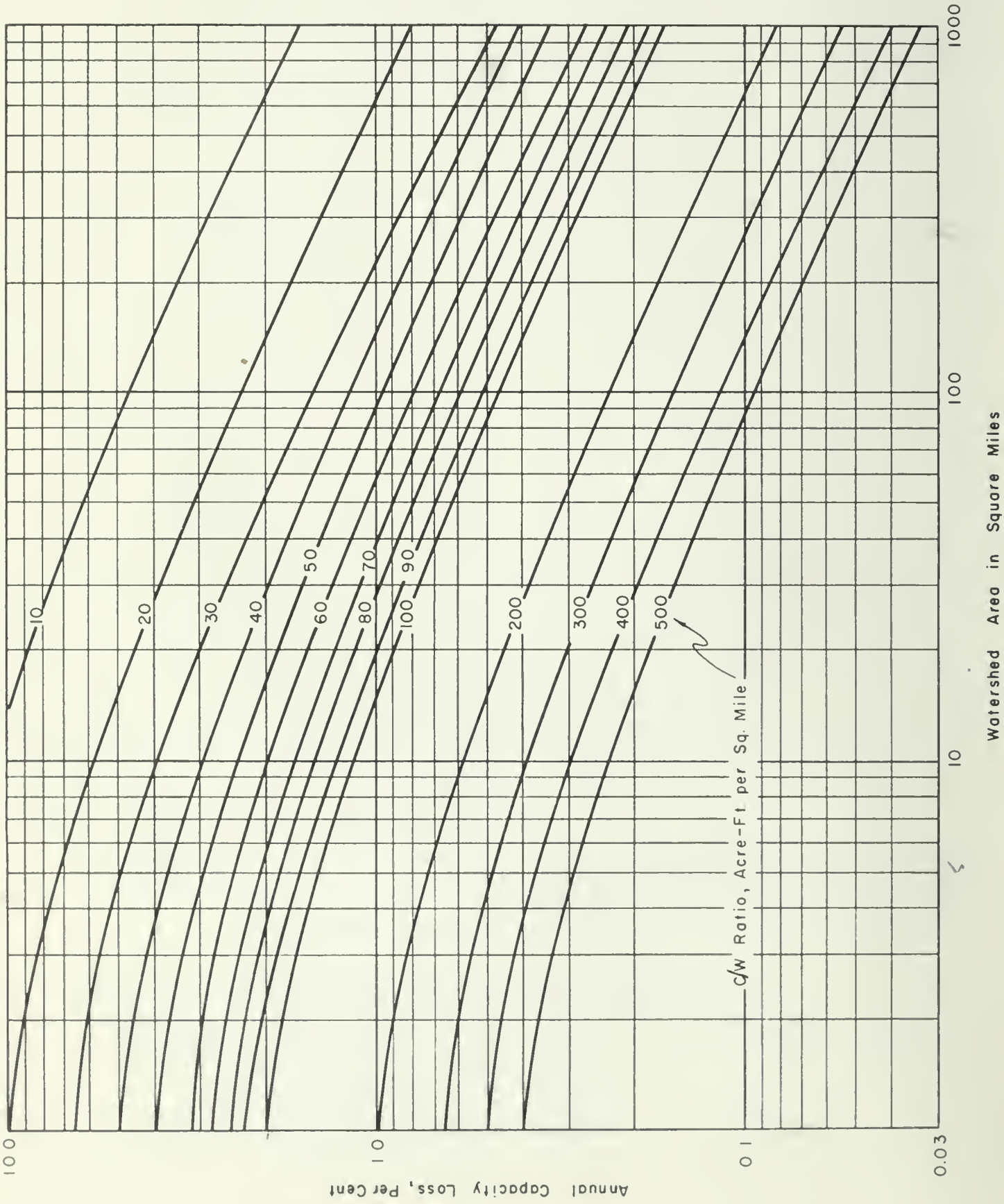


FIGURE 43. LOSS OF CAPACITY RATES DUE TO SEDIMENTATION (ILLINOIS DATA).

"trap efficiency"[34]. Reservoirs with C/W ratios below 20 may not halt the majority of the sediment that reaches them.

The data collected from sedimentation surveys in Illinois have been summarized in Figure 43 using a scheme suggested by L. C. Gottschalk[35]. This graph may be entered with the value of watershed area that is under consideration. Proceeding vertically along that value to the appropriate C/W value, the rate of capacity loss due to sedimentation may be estimated in per cent per year. A preliminary analysis of the data on sedimentation for 20 impounding reservoirs in Illinois showed the median departure from these estimates to be less than 50 per cent. Greater departures may occasionally occur, as is shown in the original publication of the data[33].

Studies now in progress by the State Water Survey in cooperation with the University of Illinois Agricultural Experiment Station, and the Agricultural Research Service and the Soil Conservation Service of the U. S. Department of Agriculture indicate that rates of sedimentation in reservoirs are dependent on a large number of factors, among which are rainfall intensities and frequencies, soil characteristics, land use, nature of drainage network, slopes, farming practices, drainage area, and reservoir storage capacity. Until these studies are more complete, it is not possible to give a summary statement of the effect of each factor, but present data have enabled estimation of the relationship of rates of sedimentation in reservoirs on the basis of the drainage area and reservoir storage capacity.

The effects of sedimentation losses on capabilities of reservoirs are less simple than they would appear. One would expect to compute the reservoir storage requirements for a chosen life of reservoir, an estimated future demand, and a particular low-flow frequency. To the storage required to meet these parameters, would be added the storage space necessary to provide for sedimentation.

Under such a program, the reservoir would be adequate to meet all conditions until the end of the chosen life. However, because of the initial provision of sediment storage space and because of the lack of immediate attainment of the anticipated demand, the reservoir would, from the time of construction until immediately prior to the end of its life, have excess capacity. In effect, this system of design adds a factor of safety which is present but diminishing until, at the end of the life of the reservoir, the factor of safety is unity.

There is reason to believe that the loss of storage capacity in reservoirs owing to sedimentation occurs in a step-wise fashion, with major increments of sediment entering the reservoir during infrequent periods of very high runoff. For design purposes, lacking information on incidence of future periods of major runoff, it is necessary to assume that the rate of sedimentation will be annually uniform.

As a result of the above considerations, it would appear desirable (a) to design the reservoir for a chosen life, and for certain other chosen parameters, and then (b) to review the situation of the reservoir with regard to sediment damage and changes in demand after approximately one-half the estimated life span of the reservoir. Such a practice would permit advance planning to take care of unusual situations such as accelerated erosion, unexpected increases in demand, or changes in streamflow characteristics.

Evaporative Losses

An additional factor that must be evaluated in reservoir design is the loss of water through evaporation. Evaporation losses may be said to consist of "gross evaporation" which is the total loss of water from the water surface to the atmosphere, and "net evaporative loss" which is the gross evaporation minus the rainfall on the water surface.

Gross Evaporation. Data on evaporative losses from standard Weather Bureau Class A pans in Illinois have been published[36] These data yield values on rates of evaporation only for the warmer months, and further work has therefore been

TABLE 11

EVAPORATION RATES AT CARBONDALE AND URBANA

Month	Rate of Evaporation in Inches per Month			
	Pan and Evaporimeter Data Carbondale and Urbana Stations		Data from Meyer	
	Carbondale	Urbana	Carbondale	Urbana
January	2.48	1.2	0.84	0.63
February	2.52	1.20	1.07	0.80
March	3.31	2.51	1.68	1.25
April	4.96	3.88	3.05	2.30
May	6.06	5.25	3.95	3.20
June	7.05	6.32	5.20	4.50
July	7.11	6.58	6.80	6.10
August	6.37	5.55	6.00	5.50
September	5.04	4.28	5.05	4.60
October	3.18	3.34	3.71	2.90
November	.90	1.96	2.35	1.75
December	.62	1.58	1.10	0.60
Total	49.60	43.57	40.80	34.13

done through the development of an evaporimeter, which records losses of water during the colder months[37]. Combining these two sets of data, evaporation values by months can be obtained throughout the year. Table 11 presents the measurements at Urbana and Carbondale for the period April 1 to September 30 for the Class A pans, and October 1 to March 31 for the evaporimeters. The pan data are for the period 1947-54, and the evaporimeter data are for 1953-55. For purposes of comparison, values computed by Meyer[38] have also been tabulated for the same locations. The values taken from Meyer are average values for evaporation from lakes, while the pan measurements give values that are higher than actually occur from lakes. It is customary to reduce pan measurement values by 30 per cent in applying them to lakes [39] [40]. Evaporation data for Springfield were not used because they do not appear concordant with those for other nearby stations.

It may be seen from Table 11 that there are large seasonal variations in evaporation rates. Inspection of the data and comparison with Figure 7 indicate that these maximum rates of evaporation coincide with the annual periods of minimum streamflow. The Illinois data are not sufficient to determine annual variations in gross evaporation. It therefore appears advisable to estimate gross evaporation by a conservative method in order to take into account the possible occurrence of high gross evaporation rates during years of extremely low streamflow. This method will, of course, be subject to considerable refinement as evaporation data are improved.

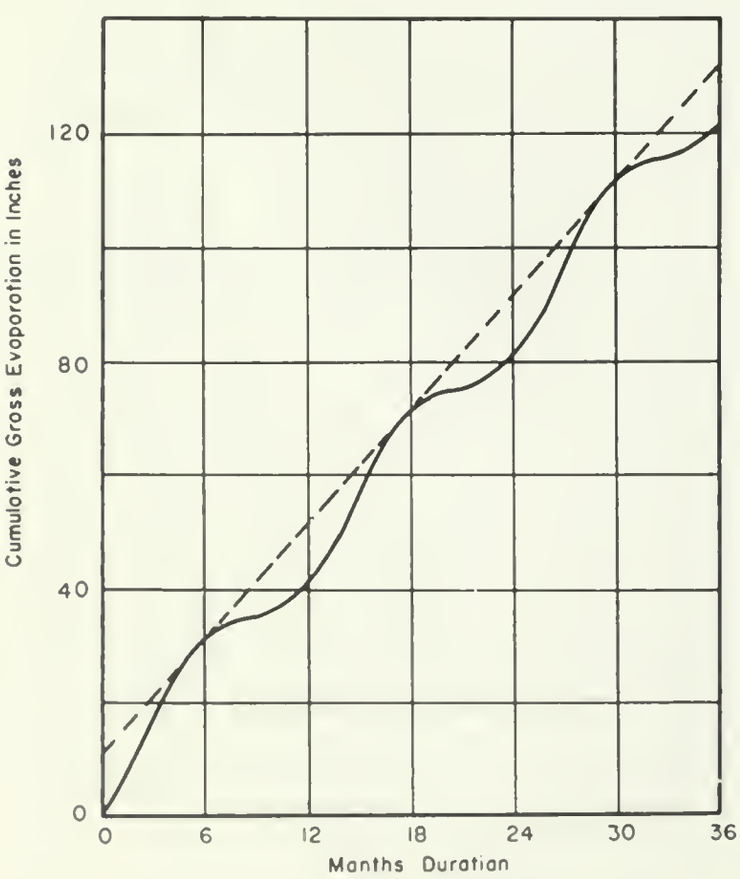


FIGURE 44. CUMULATIVE EVAPORATION AT VANDALIA.

A mass curve presentation of evaporation rate data, assuming equal monthly rates of evaporation for each of several successive years has been prepared, as shown in Figure 44. This curve is based on evaporation from lakes estimated by Meyer's method for the vicinity of Vandalia, which is approximately in the center of the area covered by this report. From this curve it may be seen that the most critical duration periods for evaporation losses are 6, 18, and 30 months. For the intervening periods, less rigorous values of evaporation would apply. It is believed desirable to ignore these less rigorous values in order to make allowance for variations in evaporation rates that may occur from year to year, and which may occur during a period of extreme flow. Accordingly, the dashed line in Figure 44 has been constructed to define evaporation losses for various duration periods.

Evaporation data for Carbondale, Quincy, Springfield, Urbana and Vandalia estimated from the maps of Meyer were studied. In each case it was clear that the maximum six months' evaporation occurred in the period from April through October. For estimating the most extreme values of evaporation that might be obtained during any duration period, the following formula yields conservative results:

$A = B + CD$, in which

A = Total gross evaporation for the duration period, inches

B = The maximum gross evaporation for the area under study for a six-month period, less 1/2 the annual gross evaporation, inches

C = Annual gross evaporation, inches, and

D = Duration period in years

Values of B and C are given in Table 12.

TABLE 12

ESTIMATED GROSS EVAPORATION RATES

Location	Average Annual Precipitation Inches	Max. 6-Month Evaporation in Inches	Max. 6-Month Evaporation Minus 1/2 Annual, Inches (B)	Annual Evaporation in Inches (C)
Carbondale	43.45	30.9	10.5	40.8
Quincy	35.86	30.6	11.2	38.8
Springfield	36.45	30.6	11.3	38.6
Urbana	36.05	28.9	10.6	36.6
Vandalia	38.36	30.8	10.9	39.8

Area Exposed to Evaporation. A review of the literature indicates that no simple but comprehensive method has been published for taking into account evaporative losses from reservoirs prior to the detailed computation of losses for each particular proposed project. At the time that Allen Hazen published his original studies on storage to be provided in impounding reservoirs [7], allowances for evaporation were discussed by Marsh and others [41]. Hazen had estimated that evaporation losses should be calculated on a reservoir surface area which was 90 per cent of the area of the reservoir when full. Marsh suggested using the water area corresponding to a storage of two-thirds of the maximum quantity stored. Cory [42], suggested that evaporation should be considered as one part of the total draft, just as net draft is the other part. Cory did not suggest any method for accomplishing this, however.

Hazen found no way of accomplishing Cory's suggestion of including evaporation in the draft, owing to the variability in reservoir characteristics and in net evaporation losses from one location to another.

In the discussion of the Hazen paper the topic of relation of water area to land area is also analyzed. Several of the discussers point out that, when the lake has a very large surface area in comparison to the total watershed area, a reduction in estimated runoff is necessary in order to compensate for the reduction in available land surface. This is to some extent offset by gains due to rainfall on the lake surface, but where evaporation exceeds annual precipitation, a net loss may be obtained by making the lake larger. This influence has been ignored in these studies as being a minor one.

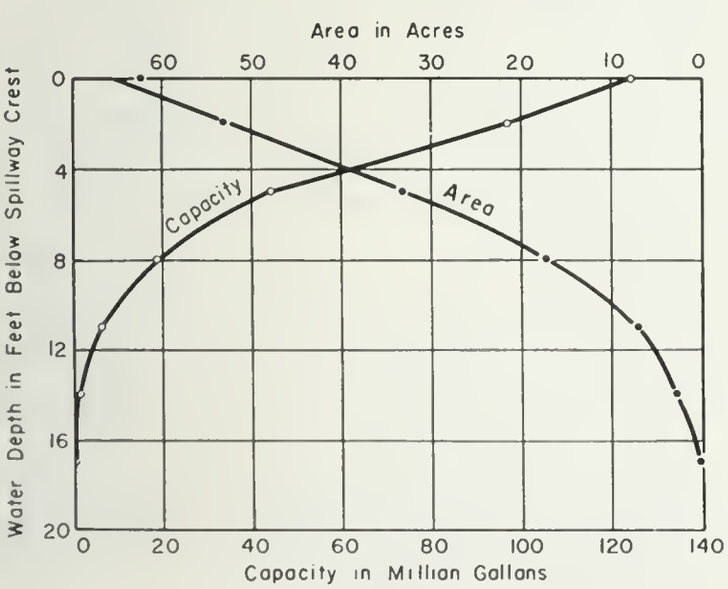


FIGURE 45. AREA AND CAPACITY CHARACTERISTICS.
Johnston City Reservoir.

It is expected that, during the design low-flow period, a reservoir will recede (at the design draft rate) from spillway level to a completely empty state at the end of the low-flow period. For purposes of simplification it may be assumed that, except at the end of the low-flow period, inflow into the reservoir will be immaterial, and that withdrawals of water from the reservoir will take place at a uniform rate during the entire period of recession. Thus the rate of removal of water from storage will be uniform with respect to time.

Figure 45 shows a typical set of area-depth and depth-capacity curves for a water supply reservoir. From such a pair of curves an "area-capacity" curve may be constructed. Area-capacity curves have been constructed for 15 municipal water supply reservoirs in Illinois which were surveyed in 1954 for which area-depth and depth-capacity curves were prepared. Areas and capacities have been expressed in per cent of the values obtained when the reservoirs were full. The reservoirs used for this determination and their characteristics are listed in Table 13. The results are summarized in Figure 46.

TABLE 13

CHARACTERISTICS OF RESERVOIRS SURVEYED IN 1954

Community	Year of Construction	Capacity, Million Gallons	Surface Area, Acres	Average Depth, Feet	Watershed Area, Square Miles
Ashley	1941	45.0	18.0	7.6	1.24
Bunker Hill	1937	12.0	17.0	2.2	7.19
Coulterville	1940	64.7	26.7	7.4	1.22
DuQuoin	1937	543.5	244	6.8	10.73
Gillespie	1923	226	70.3	9.8	5.73
Johnston City	1922	119	65.5	5.6	3.85
Nashville	1936	94	39.5	7.3	1.39
Norris City	1937	39.9	15.7	7.8	0.83
Oakland	1937	32.7	25.0	4.0	14.3
St. Elmo	1935	39.4	20.0	6.0	3.0
C.&E.I. (St. Elmo)	Prior to 1934	7.4	9.0	2.5	2.93
Staunton	1926	371	89.6	12.7	3.68
Virginia	1933	36.9	18.9	6.0	0.828
Waverly	1938	78.9	38.9	6.2	13.8
White Hall	1897	112.2	37.2	9.2	0.97

The solid line in Figure 46 represents the arithmetic mean of area values determined for capacity values of 20, 40, 60 and 80 per cent of full capacity. The dashed lines in the figure were drawn through the extreme values obtained from the 15 reservoirs used. The departures (of

area) from the mean curve were small except in the case of the area for 80 per cent of capacity, for which one value (67 per cent) for one of the lakes fell well outside the envelope curve. It will be noted by inspection of the curves in Figure 46 that departures from the mean curve generally did not exceed 10 percentage points. This concordance of the data indicates that there is a general similarity in the geometry of reservoir sites in the areas studied, and that it is therefore reasonable to use the mean curve for purposes of estimating the effective area exposed to evaporative action.

The area under the solid curve in Figure 46, divided by 100, will equal the mean surface area with respect to time when draft from the reservoir is continuous and uniform. The mean value of this area was found to be 64.4 per cent of the surface area with water level at spillway crest. The 15 cases studied indicate that values ranging from 55 to 70 per cent may occur. This method of approach makes no allowance for changes in surface area produced by sedimentation with time.

It is concluded that, for southern Illinois water supply reservoirs, it would be reasonable to apply evaporation estimates to an area which is 64 per cent of the design surface area of the reservoir.

The surface area of a reservoir is equal to the volume divided by its average depth. Table 3 gives data on the average depth of 40 impounding reservoirs for municipal water supplies in

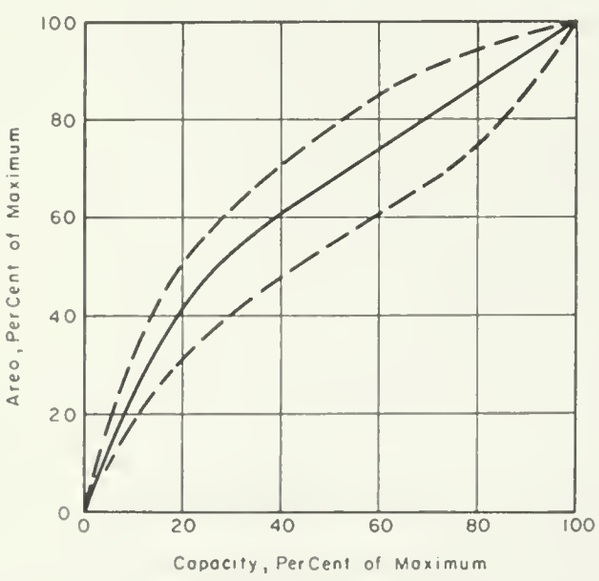


FIGURE 46. AREA-CAPACITY RELATIONSHIPS FOR ILLINOIS RESERVOIRS. 1954 Area-Depth Curves.

Illinois. These figures were obtained by dividing the original total capacity of the reservoir by the original surface area. It was noted that the median of the average depths of these reservoirs is 8.7 feet, and that the extreme values are 4 feet and 20 feet.

While most of the reservoirs studied were small, detailed examination of the data for the two larger ones (DuQuoin and Staunton) showed them to be in close agreement with the average curve given in Figure 46. Additional data on characteristics of reservoirs in Illinois were assembled in Table 14, covering the largest reservoirs in the State. Area and capacity-depth curves are not available for all of these reservoirs, but the data given indicate that their average depths are not greatly different from those of the smaller reservoirs shown in Table 13. The average depths of the larger reservoirs are greater than those of the smaller ones. In estimating average depth for a reservoir, it would therefore be desirable to proceed on the basis that the average depth of a reservoir storing more than one billion gallons could very well be of the order of 15 feet, while the average depth for reservoirs storing 500 million gallons or less might be 10 feet or less.

TABLE 14

CHARACTERISTICS OF LARGE IMPOUNDING RESERVOIRS IN ILLINOIS

	Year of Construction	Capacity, Million Gallons	Surface Area, Acres	Average Depth, Feet	Watershed Area, Square Miles
Bracken Near Galesburg	1923	802	181	14.5	8.91
Bloomington Near Bloomington	1929	1,927	487.2	13.2	61
Carlinville Near Carlinville	1939	465	167	9.1	26.1
Charleston Near Charleston	1947	860	400	7.0	800
Crab Orchard Near Carterville	1940	21,938	6,965	10.3	196
Decatur Near Decatur	1922	4,763	2,604	6.0	906
Hillsboro Near Hillsboro	1918	500	96	17.0	7.5
Jacksonville Near Jacksonville	1939	2,240	469	15.6	10.8
Mauvaise Terre Near Jacksonville	1921	396	223	5.8	32.6
Little Grassy Near Makanda	1942	8,417	1,000	27.5	15.1
Springfield Near Springfield	1934	19,089	4,234	14.7	258
Vermilion Near Danville	1902	2,600	900	9.5	267

NOTE: Capacity and area values based on most recent survey data.

Average depth data may be used for making preliminary estimates of reservoir surface areas. For the region studied, it would appear desirable to calculate such areas for average depths of 5 to 20 feet. Data covering this range of average depth would take into account variances in topography from place to place. At some locations

reservoirs with small average depths may be necessary, and at others, greater average depths may be attained. These variations depend on the topography and on dam height. Inasmuch as this study indicates a need for more storage than has hitherto been provided, average depths of 10 feet or more may be expected in future reservoirs.

To sum up, the evaporative losses may be seen to be a function of the average depth of reservoirs. When the storage volume is known, use of an average depth figure enables estimation of the surface area of the reservoir. To this may be applied the factor 0.644. To the resulting reduced area value, may be applied net evaporative loss rates (discussed below). As will be pointed out, this process can be combined with the process of estimating storage for any stated draft rate.

Net Evaporative Rates. In order to obtain net evaporation rate values, appropriate values for rainfall on the water surface must be subtracted from the values of gross evaporation. Such rainfall values, for various duration periods and for various frequencies of occurrence have been prepared and are given elsewhere (Figure 32). It would be convenient if it should turn out that, throughout a given region, net evaporative loss rates were uniform. Studies were made of calculated net evaporative loss rates at Quincy, Springfield, Urbana, Vandalia, and Carbondale. These studies indicated that the evaporation and precipitation patterns in the southern half of Illinois are sufficiently different that net evaporative loss values are not uniform throughout central and southern Illinois. Departures of as much as 20 per cent from a mean curve were noted, but the mean for each of five different dry-weather frequency periods tested was quite well represented by the data for the region of Vandalia. The Vandalia values for net evaporative loss appeared to check those for the other locations within five per cent for duration periods up to two years, and the more severe errors occurred for the 2 1/2 and 3-year duration periods.

The computed results for the vicinity of Vandalia are shown in Figure 47, which gives the net loss due to evaporation for dry periods of various durations and frequencies.

Net Yield of Reservoirs

In the preceding material on reservoir capabilities, storage requirements have been related to the draft rate on the reservoir. The draft rate is the total withdrawal from the reservoir, and includes (a) water supply requirements, (b) evaporative losses and (c) seepage losses. No information is available for determination of seepage losses quantitatively at this time. Attempts to secure evaluations of seepage loss have indicated that this quantity, with properly constructed dams, is negligible.

It would obviously be desirable to be able to estimate net yield of reservoirs by taking into account evaporative losses as well as water supply requirements. This may be done by estimating the lake surface areas exposed to evaporation, and applying appropriate evaporation rates to the area values. Draft rates are then reduced by the amount of the evaporative loss (which may be expressed in per cent of mean flow rate and called the evaporative draft rate) to obtain "net yield" values.

In calculating draft rates, consideration should be given to the demand during the low-flow duration period that is expected to control the reservoir design. For low-flow periods in excess of one year, it would seem proper to use the average annual water requirement from the lake plus net evaporative loss to estimate draft. For those situations in which short low-flow duration periods are expected to control or in which unusual variations in demand are expected, it may be desirable to use (a) a draft rate based upon summertime pumping rates plus summertime evaporation losses or (b) a draft rate based on other special local conditions. Inasmuch as those situations in which short-term low-flow periods may control will probably be avoided through the desire to reduce sediment damage, in general the average annual withdrawal rate may be used.

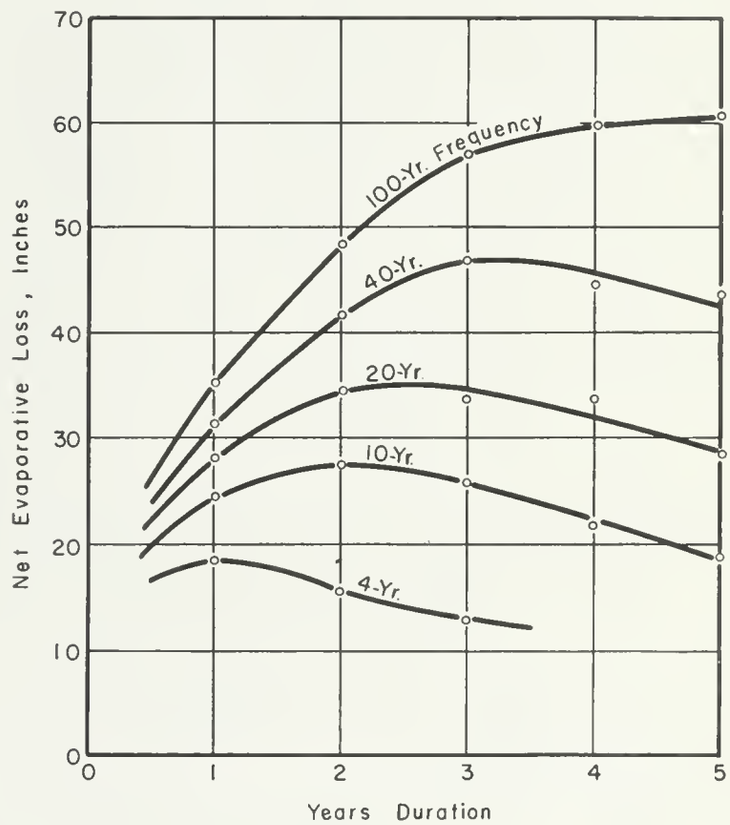


FIGURE 47. ESTIMATED NET EVAPORATIVE LOSSES FOR PERIODS OF LOW RAINFALL OF VARIOUS FREQUENCIES AND DURATIONS.

For the calculation of net yield, it is necessary to make correction for evaporative losses

for fixed duration periods. Accordingly, from Figure 42, values of storage and draft rate (expressed in per cent of mean annual flow) were tabulated for 1, 2, 3, 4 and 5-year duration periods for 4, 10, 20, 40 and 100-year recurrence intervals.

For the computations of net yield that follow, mean annual streamflow in the area was taken as 10 inches per year. Storage volume values were converted into acre-feet. These values - divided by an assumed average depth for the reservoir of 10 feet - yielded reservoir surface area

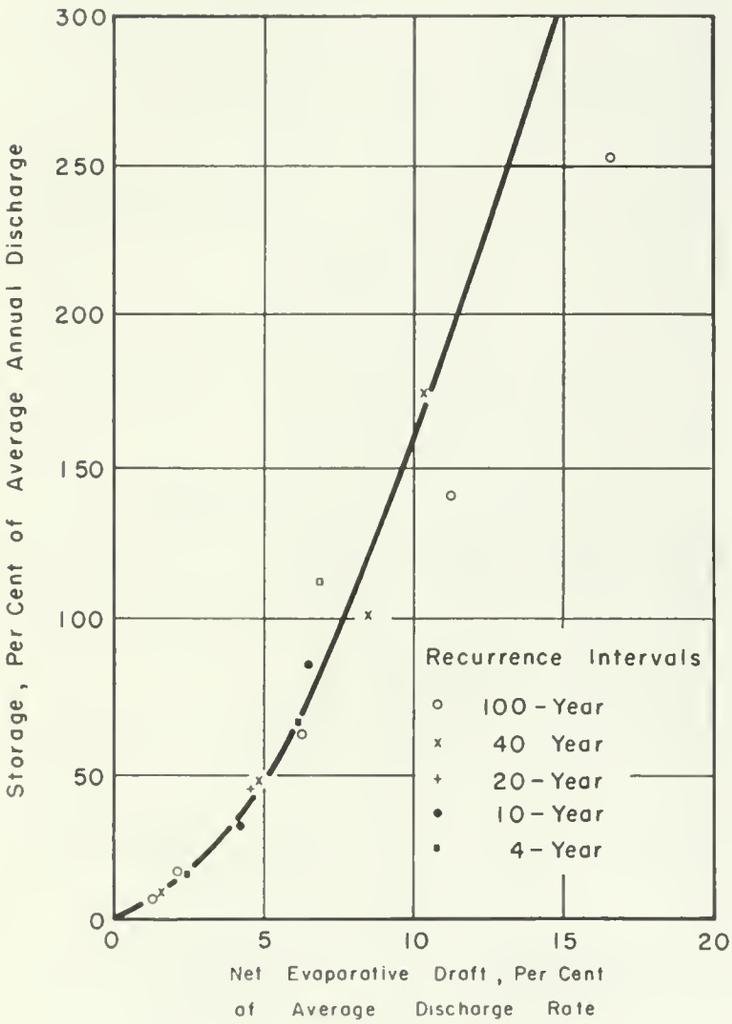


FIGURE 48. NET EVAPORATIVE DRAFT FROM RESERVOIRS.

values in acres. Each of these was reduced to 64.4 per cent of its value in accordance with the findings above. Appropriate values for net evaporative loss obtained from Figure 47 were then applied to obtain the evaporative drafts for each of the surface area values. These were then expressed in per cent of mean flow, producing values of the "net evaporative draft rate", which may be defined as the ratio, expressed as a per cent of the net evaporative loss rate to the mean discharge rate for the site under consideration. Actually this process does not require consideration of the mean streamflow, and the same net evaporative draft rate values, expressed in per cent of mean flow, are obtained regardless of the streamflow assumed.

It was found that, for the location studied, there appears to be an approximate general relationship between storage volume and the net evaporative draft rate from the reservoir. The data establishing this relationship are shown in Figure 48 for a 10-foot average depth of reservoir. For any other average depth, these values may be increased or reduced in inverse proportion to the average depth. The adjusted values of net evaporative draft rate may then be subtracted from the draft rate values obtained from Figure 42 to obtain values of "net yield" in per cent of mean streamflow.

The values given in Figure 48 are for the vicinity of Vandalia and, while they are probably fairly trustworthy throughout southern Illinois, should not be relied on for design computations elsewhere, without checking them against local data. The correlation between storage volume and net evaporative draft should also be checked before application to other locations.

From Figure 48 it will be noted that, except for very high storage volumes, the net evaporative draft rate is relatively small. In no case does it reach 20 per cent of the mean discharge rate. The most serious effects of the net evaporative draft rate will be felt in those situations which require high storage values, especially when particularly shallow lakes are necessary.

To complete the portrayal of effects of net evaporative losses, Figure 49 has been prepared. It shows the relationship between net yield and storage required, assuming an average depth of 10 feet. For any particular site, a different relationship would hold, since average depth would vary with reservoir capacity.

Limitation of Method

Although the results obtained in this study have been generalized, it is believed that they will yield satisfactory approximations for preliminary evaluation of reservoirs in the southern half of Illinois. The results obtained should not be regarded as a final basis for design of any specific reservoir. For design, there is no substitute for thoughtful, intensive analysis of hydrologic records collected in the vicinity of the proposed project. The designer may find the methods used in preparing this study helpful in analysis of local records for design.

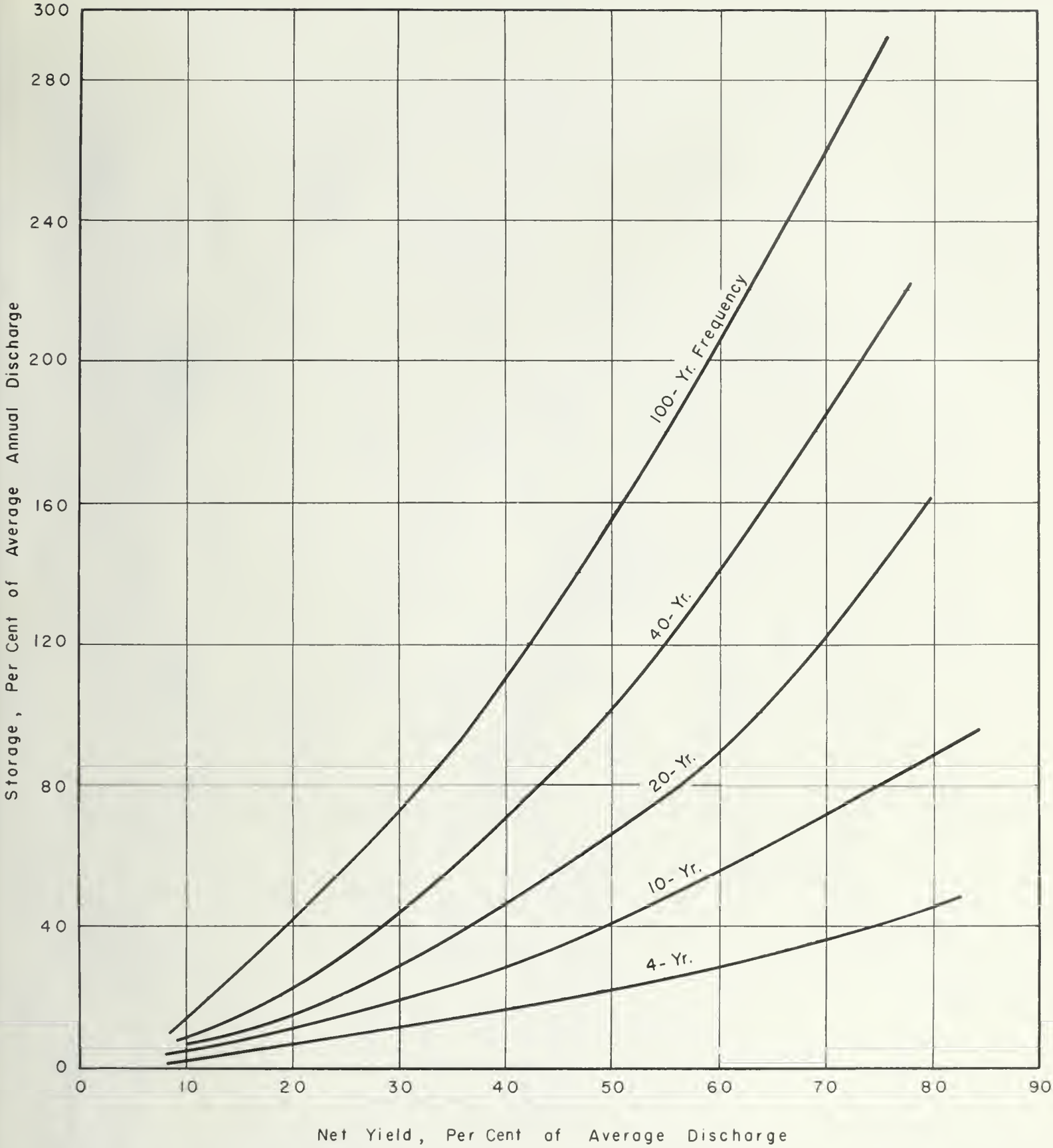
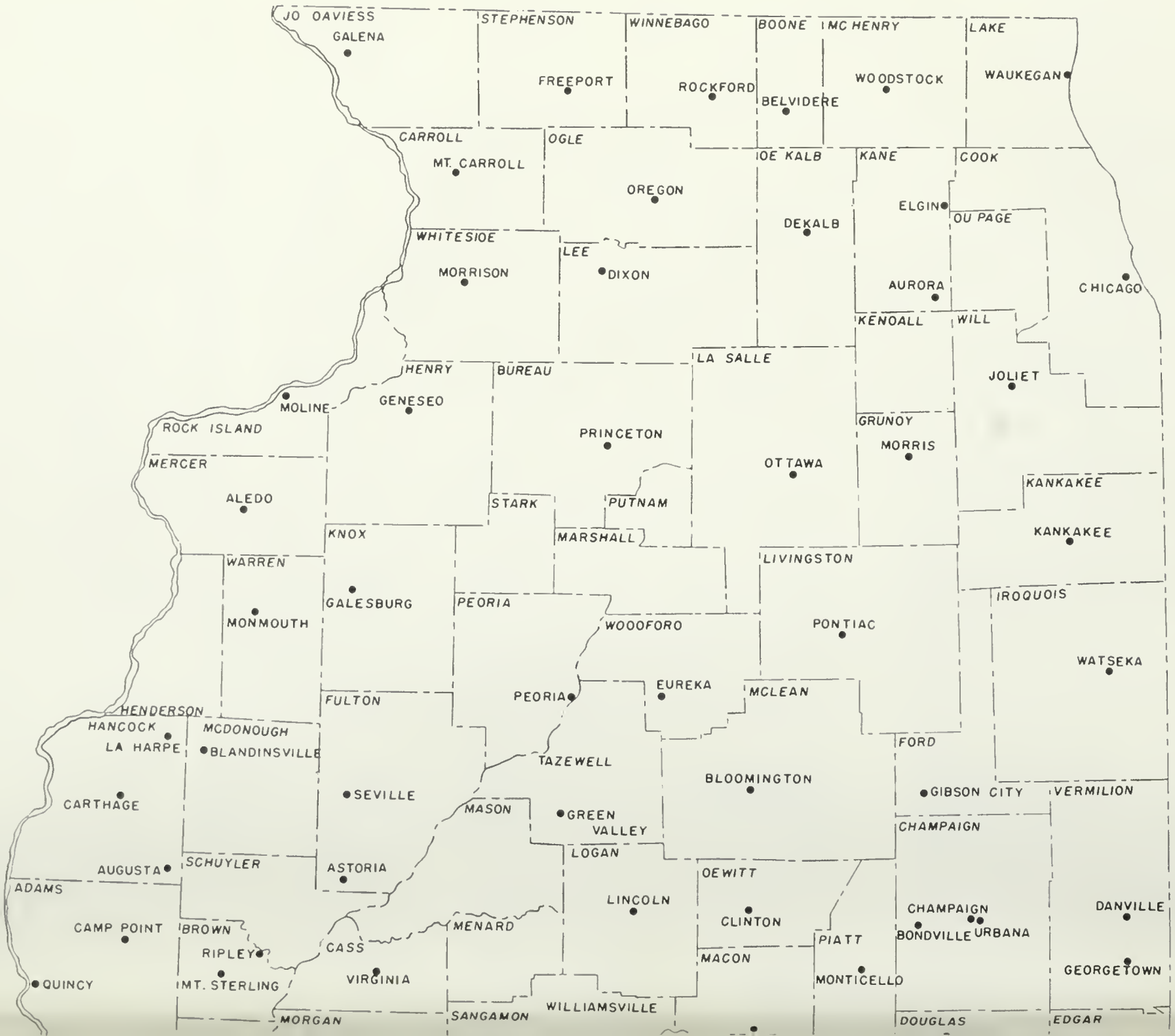


FIGURE 49. NET YIELD OF RESERVOIRS HAVING 10-FOOT AVERAGE DEPTH, Southern Illinois.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FREEPORT
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WAUKEGAN
CARROLL
MT. CARROLL
OGLE
OREGON
DE KALB
DEKALB
KANE
ELGIN
COOK
DU PAGE
WHITESIDE
MORRISON
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DIXON
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RIPLEY
CASS
MACON
GEORGETOWN
QUINCY
MT. STERLING
VIRGINIA
MONTICELLO
WILLIAMSVILLE
MORGAN
SANGAMON
DOUGLAS
EDGAR

FIGURE 50. ORIENTATION MAP OF ILLINOIS.

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DEPARTMENT OF REGISTRATION AND EDUCATION
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RAINFALL RELATIONS ON SMALL AREAS IN ILLINOIS

BY

F. A. HUFF AND J. C. NEILL

ILLINOIS STATE WATER SURVEY
WILLIAM C. ACKERMANN, Chief

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INTRODUCTION

During the period 1948-55, the State Water Survey Meteorology Subdivision operated several concentrated rain gage networks on small areas to collect detailed precipitation data which can further our knowledge of the water resources of Illinois. The collected data have been used in a number of studies to obtain information on precipitation pertinent to hydrologic analysis, design, and planning. This bulletin presents current results of several of these studies. Some of the studies are of a continuing nature and as additional data become available, refinements will be published. However, it is felt that adequate analysis has now been completed to present results that may be beneficial to hydrologists and engineers actively engaged in the field of water resources.

Although these studies are based on Illinois data, the results should be approximately representative of conditions in the Midwest and of other areas having similar climate and topography. The studies to date have been concentrated on the analysis of rainfall during the spring to fall thunderstorm season on relatively small areas ranging from less than one square mile to 400 square miles. The extreme variability in precipitation experienced during the thunderstorm season creates difficult problems for the hydrologist concerned with small watersheds, which are subject to flash floods and rapidly affected by drought conditions. The major portion of the surface-water supplies for municipalities in the state are obtained from lakes having watershed areas under 400 square miles.

Included in this bulletin are results of studies on: the relative variability of storm and monthly rainfall over small areas: the distribution of point and areal mean rainfall rates in shower-type precipitation; area-depth relations on small watersheds; the variation of point rainfall with distance; the areal representativeness of point rainfall in measuring areal mean rainfall on a storm, weekly, and monthly basis; the combined effect of storm size, area, and number of rain gages on the accuracy of storm mean rainfall estimates; relations during periods of excessive rainfall over a 100-square mile basin; the relation between point and areal mean rainfall frequencies; and micro-meteorological variations in storm rainfall.

ACKNOWLEDGMENTS

This bulletin was prepared under the direction of William C. Ackermann, Chief of the Illinois State Water Survey. Research was accomplished under the general guidance of Glenn E. Stout, Head, Meteorology Subdivision.

The authors are indebted to A. M. Buswell, former Chief, and H. E. Hudson, Jr., former Head of the Engineering Subdivision, for suggestions and guidance in the early phases of the research incorporated in the bulletin.

Within the Meteorology Subdivision, special credit is due Douglas M. A. Jones, who supervised a major portion of the data collection and Carol Verseman, who performed or supervised a major portion of the routine analysis. Drafting of the illustrations and cover was done by John Wesselhoff. Numerous research assistants helped with the installation and servicing of rain gages, collection and tabulation of data, and computation.

Credit is due Lester Pfister, President of the Pfister Hybrid Corn Company, the U. S. Soil Conservation Service, and the Civil Engineering Department of the University of Illinois, who provided some of the rain gages for the studies. Operation of the Goose Creek network and data collection were sponsored in part by the Signal Corps Engineering Laboratories, Belmar, New Jersey under Contract DA-36-039 SC-42446. The Agricultural Engineering Department of the University of Illinois furnished data from its 400-acre rain-gage network.

Certain phases of the analyses were greatly expedited through use of facilities at the Statistical Service Unit and the Digital Computer Laboratory at the University of Illinois.

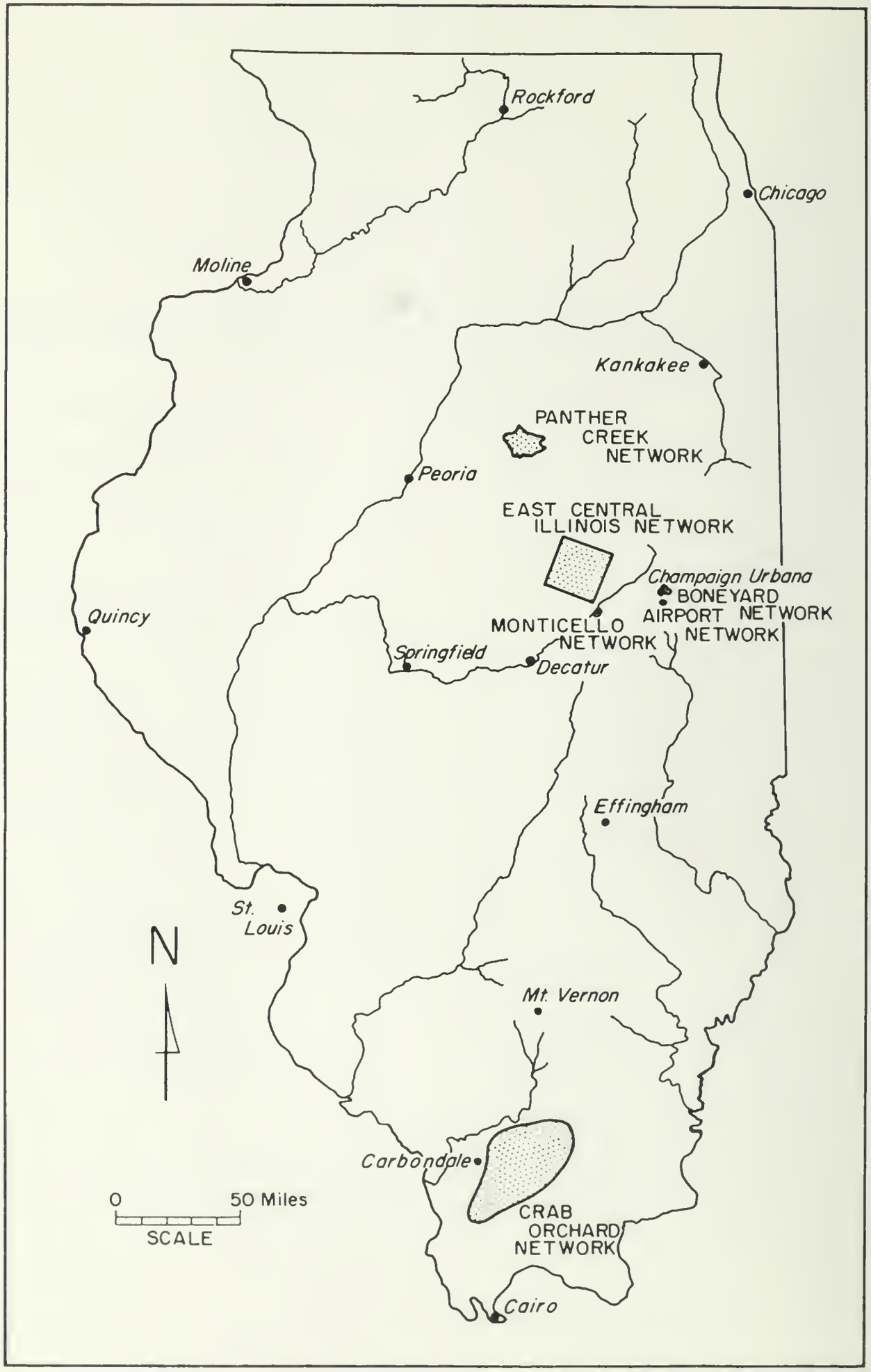


FIGURE 1 REFERENCE MAP

RAIN-GAGING FACILITIES

Rainfall data from seven rain-gage networks located in central Illinois were available for studies described in this bulletin. Generally, the networks were in full operation during the warm season from April through October, after which they were closed or reduced to relatively few gages for the winter season. The network locations within the state are shown on the reference map in Figure 1. The 1948 El Paso network which is not illustrated was a southward and westward extension of the Panther Creek network. The Goose Creek network was located in the southeastern portion of the present east central Illinois network. The Crab Orchard network in southern Illinois was not installed until the fall of 1954. Data from it have not been analyzed adequately to be presented in this bulletin.

The terrain within the central Illinois experimental areas is relatively flat with no distinct topographic features to influence storm behavior. All gages were located with open exposures. The recording gages, from which the majority of the analytical data were obtained, were serviced by trained technicians. The non-recording gages were installed by Water Survey technicians, and the measurements were taken by nearby residents who had been carefully instructed in the proper operation of the gages. In most cases, these cooperative observers were farmers having a lively interest in the measurement of rainfall.

Statistical tests were performed to determine the homogeneity of data obtained from the various networks. These tests indicated that no significant differences existed, thus permitting utilization of combined network data for the derivation of various empirical rainfall relations.

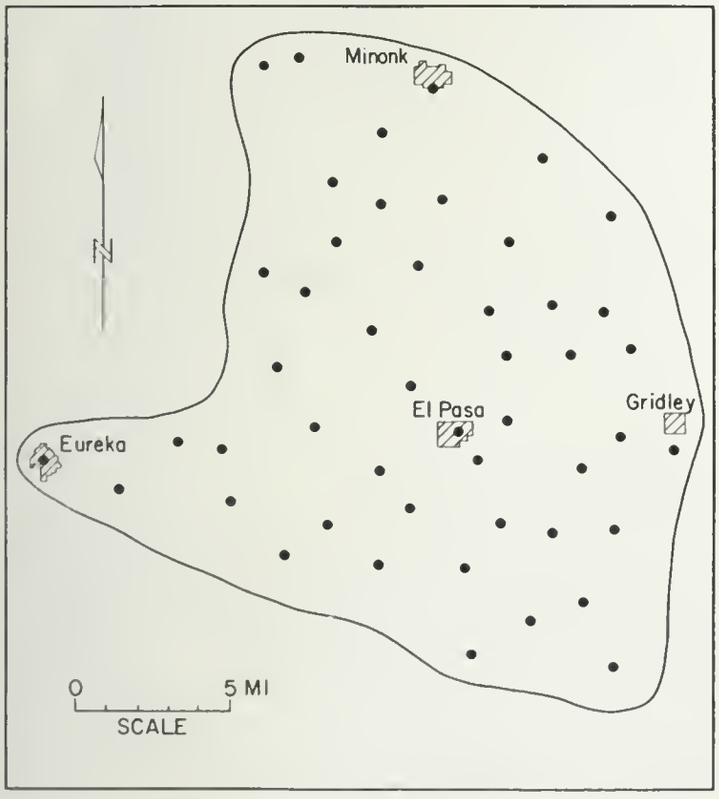


FIGURE 2 EL PASO NETWORK, 280 SQUARE MILES

El Paso Network

The first of these rain-gage networks located on an area of 280 square miles in the vicinity of El Paso, Illinois (Fig. 2) was established in the spring of 1948. During 1948 and 1949 a total of 17 recording and 31 non-recording gages, or an average of one gage per 5.8 square miles, was operated on this area.

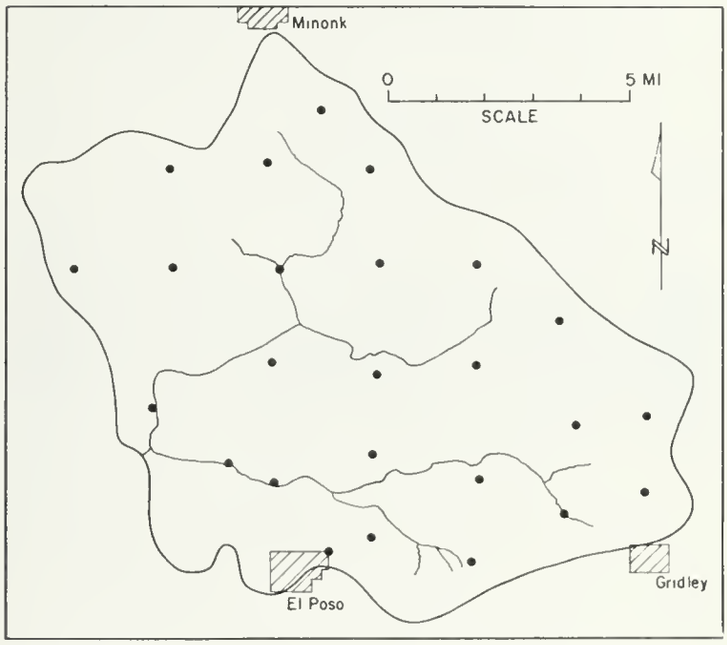


FIGURE 3 PANTHER CREEK NETWORK, 100 SQUARE MILES

Panther Creek Network

The network in the El Paso area was reduced in size at the close of the 1949 thunderstorm season. Beginning in the spring of 1950, a network of 100 square miles within the El Paso network was continued on the Panther Creek watershed. This area had been gaged with 6 recording and 14 non-recording gages during the 1948 and 1949 seasons, corresponding to an average of one gage per 4.75 square miles. Employing both recording and non-recording gages, the Panther Creek network was expanded to 40 gages in 1950. During 1951-53, the network of 25 recording gages illustrated in Figure 3 was maintained. In 1954, the Panther Creek network was reduced to 10 recording gages, since the gages were needed elsewhere and 10 was considered a sufficient number for the Panther Creek study.

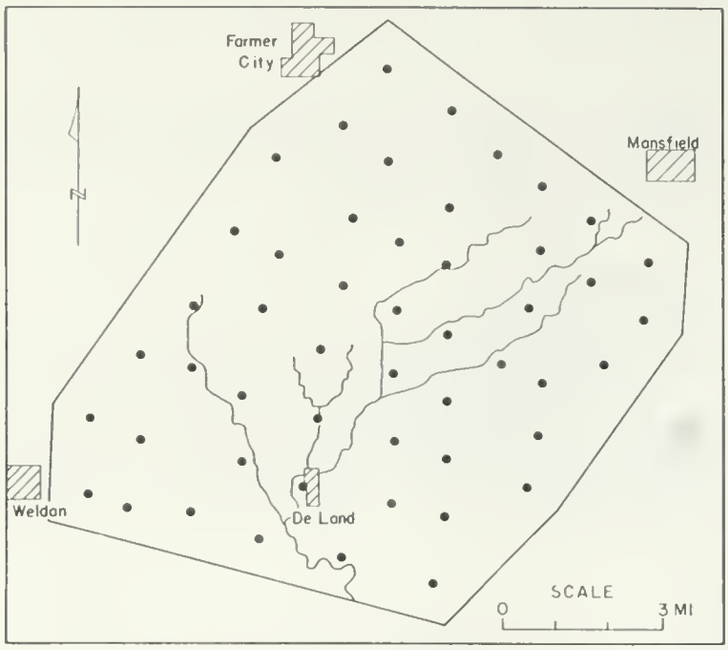


FIGURE 4 GOOSE CREEK NETWORK, 100 SQUARE MILES

Goose Creek Network

During the spring of 1951, a network was installed over an area of 50 square miles on the Goose Creek watershed in the vicinity of Deland, Illinois. This network consisted of 33 recording rain gages which were equipped with 12.65-inch diameter collectors and with chart drums making one revolution every six hours. The Goose Creek network was enlarged during July 1952 to include 100 square miles (Fig. 4). A total of 50 recording rain gages, equipped with the 12.65-inch diameter collector and the 6-hour chart drive, was operated on this area during the remainder of the 1952 thunderstorm season and during the 1953 thunderstorm season. This network was reorganized somewhat in 1954; however, the area of the network was maintained at 100 square miles. A total of 48 recording gages was used during the 1954 season.

East Central Illinois Network

In order to obtain data over an area larger than 100 square miles, the Goose creek network was reorganized and expanded north and west during the spring of 1955. The new network (Fig. 5) includes an area of 400 square miles. A total of 49 recording gages was used on this area during the 1955 thunderstorm season.

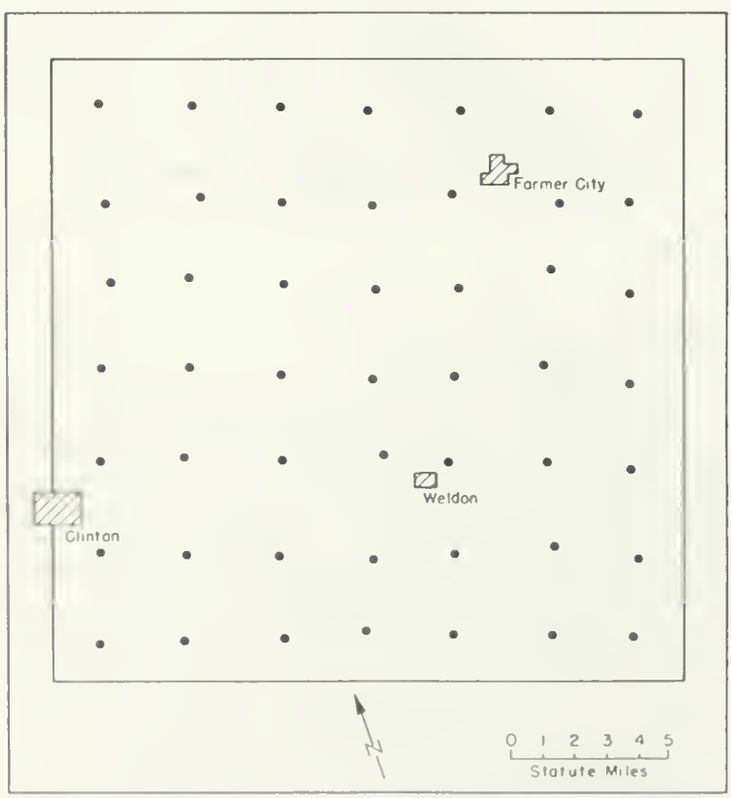


FIGURE 5 EAST CENTRAL ILLINOIS NETWORK, 400 SQUARE MILES

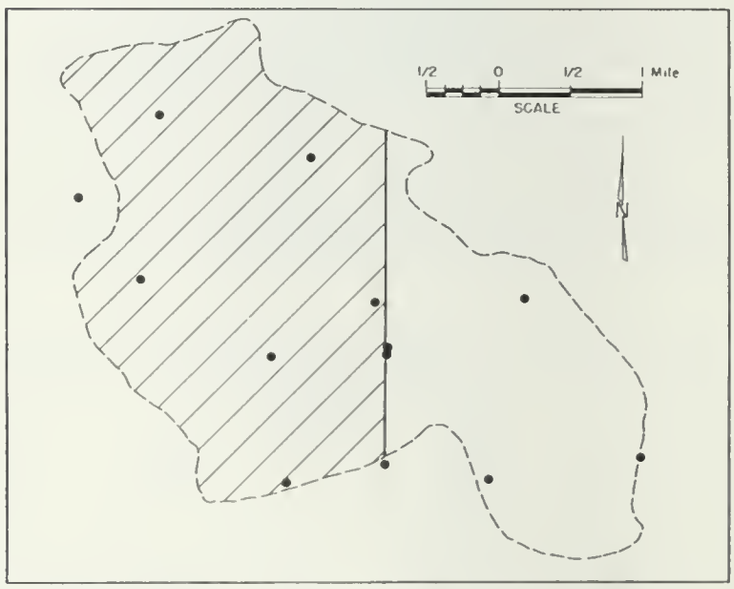


FIGURE 6 BONEYARD CREEK NETWORK, 8.5 SQUARE MILES

Boneyard Creek Network

A 12-station network for sampling an area of approximately 8.5 square miles on the Boneyard Creek watershed at Champaign-Urbana, Illinois is shown in Figure 6. This network has been operated since the fall of 1948 in cooperation with the Civil Engineering Department, University of Illinois. The majority of the rainfall analysis has been confined to data from an area of 5.5 square miles within the watershed (cross-hatched in Fig. 6). Streamflow measurements are made on this area, and the rain gage density is greater than over the remaining portion of the watershed.

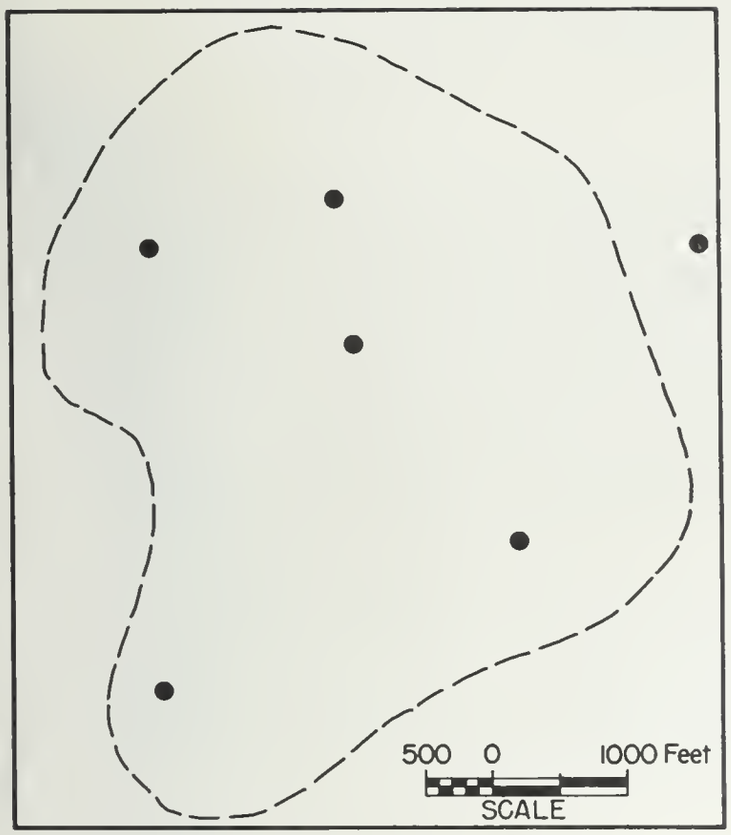


FIGURE 7 MONTICELLO NETWORK, 400 ACRES

Monticello Network

The Monticello network (Fig. 7) consisting of 400 acres (0.6 sq. mi.) is operated by the Agricultural Engineering Department, University of Illinois. For the Survey studies six recording gages were used from this network, which has been operated since 1949. Data from the network were made available to the authors for analysis and inclusion in this bulletin.

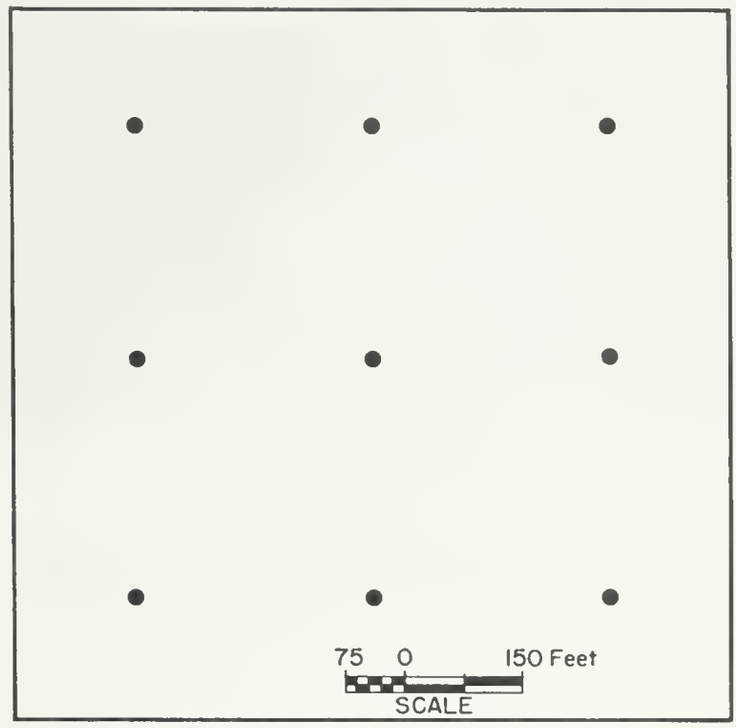


FIGURE 8 AIRPORT NETWORK, 19 ACRES

Airport Network

During 1953-55, a micro-network of 18 non-recording gages was operated at the University of Illinois Airport, six miles south of Champaign-Urbana. This network consisted of pairs of gages spaced six feet apart at each of nine stations. These stations were located at intervals of 300 feet to form a square grid pattern on 19 acres (0.03 sq. mi.) as shown in Figure 8.

RAINFALL VARIABILITY

Introduction

The great variability of thunderstorm and rainshower precipitation within relatively short distances is generally recognized by meteorologists and hydrologists. However, information concerning the degree of this variability is limited, especially for small watersheds. Obviously, rainfall variability is pertinent to hydrologists and meteorologists concerned with rain gaging of watersheds and the interpretation of rainfall data for application to hydrologic analysis. For example, the area-depth relation and, consequently, the rainfall-runoff relation are closely related to the variability within a given watershed.

Data from two concentrated networks, the 280-square mile El Paso network (Fig.2) and the 100-square mile Panther Creek network (Fig. 3) have been used for a limited study on thundershower and rainshower variability. Data for 1948-49 on the El Paso network and for 1948-53 on the Panther Creek network were used in the study.

Rainfall Relative Variability

The relative variability for these areas was obtained by calculating the coefficient of variation in each storm on each network. There were 48 gages in the El Paso network and 20 gages on the Panther Creek network used for calculations. For convenience, the variability factor (coefficient of variation) was expressed in per cent and was obtained from the following equation:

$$CV = \frac{SD}{\bar{P}} \times 100 \qquad (1)$$

where CV is the coefficient of variation in per cent, SD is the standard deviation of all the network observations, and $\bar{P}$ is the areal mean rainfall.

The variability factor is a measure of the dispersion of point rainfall values about the areal mean rainfall. In addition to the actual variations in the storm rainfall the variability factor includes the effects of instrumental or observational errors and gage non-respresentativeness. However, no distinct topographic features existed in the experimental area, and all gages were located with open exposures. Therefore, the effect of gage non-representativeness is eliminated for all practical purposes. A close check was maintained on the operation of the rain gages, so the variability data should be reasonably accurate and representative.

Results of Analysis

The data were first analyzed to determine the general effects of storm mean rainfall and area on relative variability. The results of this analysis are summarized in Tables 1 and 2. Both tables clearly indicate that for a given basin the relative variability tends to decrease with increasing storm mean rainfall. This tendency is accounted for by the nature of shower-type rainfall and the location of the heavier storm cores with respect to the basin. Considering the multi-cellular nature of thunderstorms, it appears logical that an area comes under the influence of more cells with heavy rainfall and, consequently, a tendency exists for greater relative uniformity in the rainfall pattern. It was found on the Thunderstorm Project [1] that single-celled thunderstorms were generally weak compared to the multi-cellular storms. The expected tendency for the variability to increase with area is apparent from a comparison of average values in Tables 1 and 2. The maximum and minimum values in these tables are the extremes observed for individual storms in each mean rainfall class.

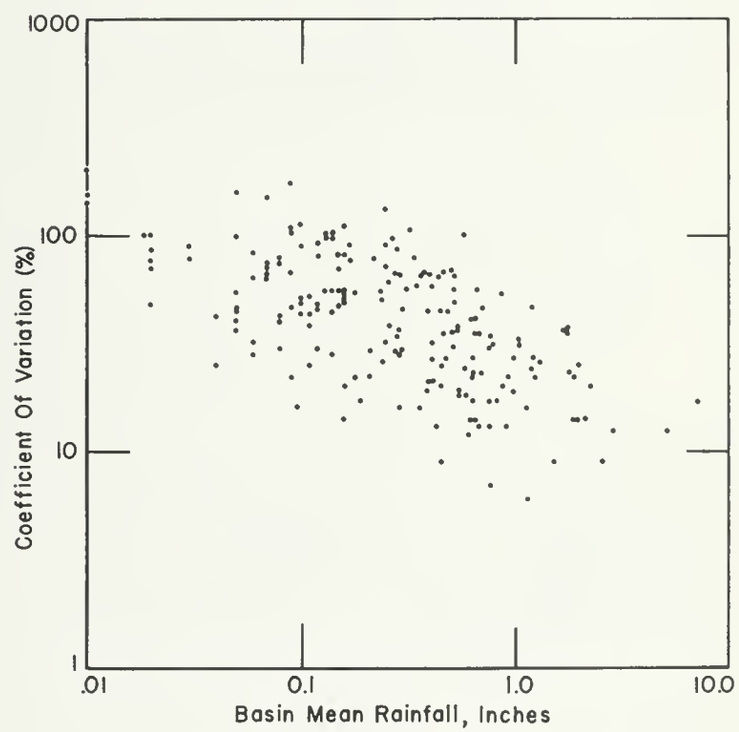


FIGURE 9 STORM RAINFALL VARIABILITY

Table 3 shows the relative variability calculated on a monthly basis for the 100-square mile drainage basin. Comparing Tables 1 and 3, it is seen that the relative variability tends to decrease as the rainfall increases and the totalizing period is increased. Figure 9 is a scatter diagram for the 100-square mile basin, illustrating the relationship between storm mean rainfall and relative variability based on 185 cases. The trend for the relative variability to decrease with increasing mean rainfall is apparent. However, a close correlation does not exist as shown by the amount of scatter on the diagram. Other factors, such as storm duration, movement of storm with respect to basin, and the internal structure of thunderstorms and rain showers undoubtedly contributed to the observed scatter.

TABLE 1

STORM RAINFALL VARIABILITY

100 SQ. MI. AREA

Mean Rainfall (in.)	Coefficient of Variation (%) Av.	Max.	Min.	Number of Cases
0.01 - 0.20	72	200	14	75
0.21 - 0.50	48	133	9	44
0.51 - 1.00	30	100	7	42
1.01 - 2.00	25	37	6	18
Over 2.00	14	20	9	6

TABLE 2

STORM RAINFALL VARIABILITY

280 SQ. MI. AREA

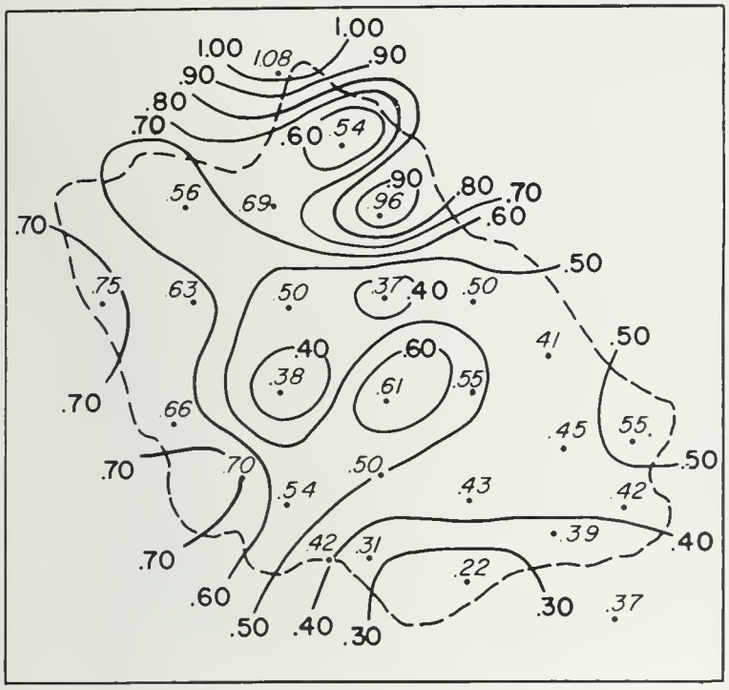
Mean Rainfall (in.)	Coefficient of Variation (%) Av.	Max.	Min.	Number of Cases
0.01 - 0.20	81	127	23	17
0.21 - 0.50	56	78	16	14
0.51 - 1.00	42	64	22	15
1.01 - 3.00	31	47	14	5

TABLE 3

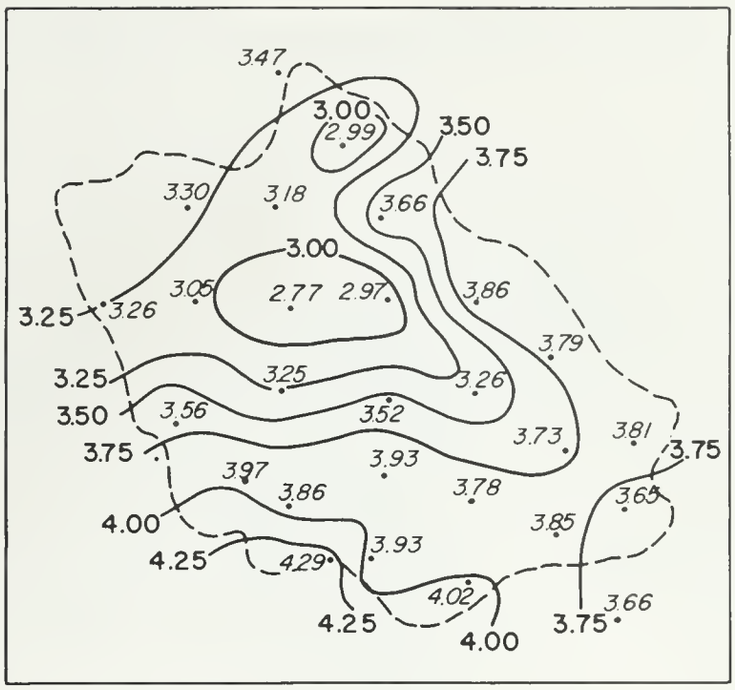
MONTHLY RAINFALL VARIABILITY

100 SQ. MI. AREA

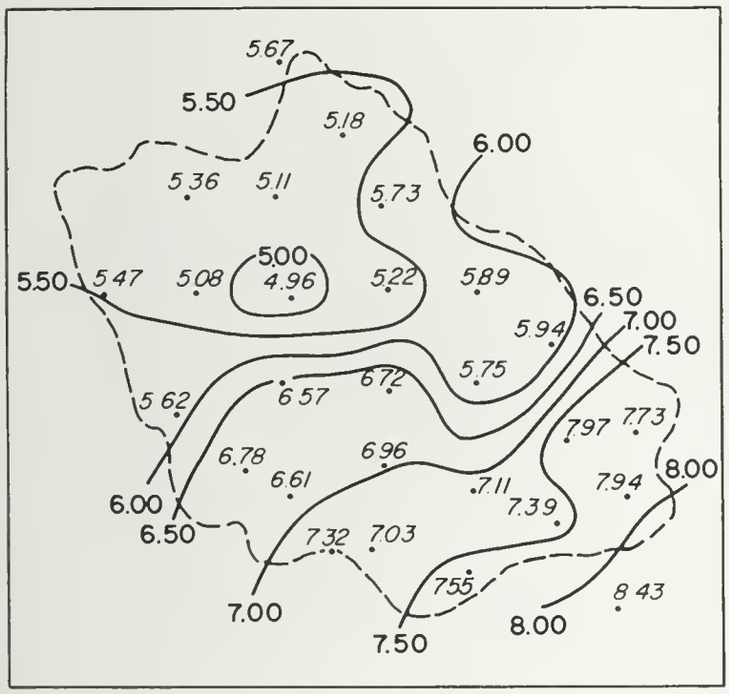
Mean Rainfall (in.)	Coefficient of Variation (%) Av.	Max.	Min.	Number of Cases
0.01 - 2.00	31	37	16	6
2.01 - 4.00	15	19	9	9
4.01 - 6.00	18	26	12	4
6.01 - 8.00	12	25	8	7



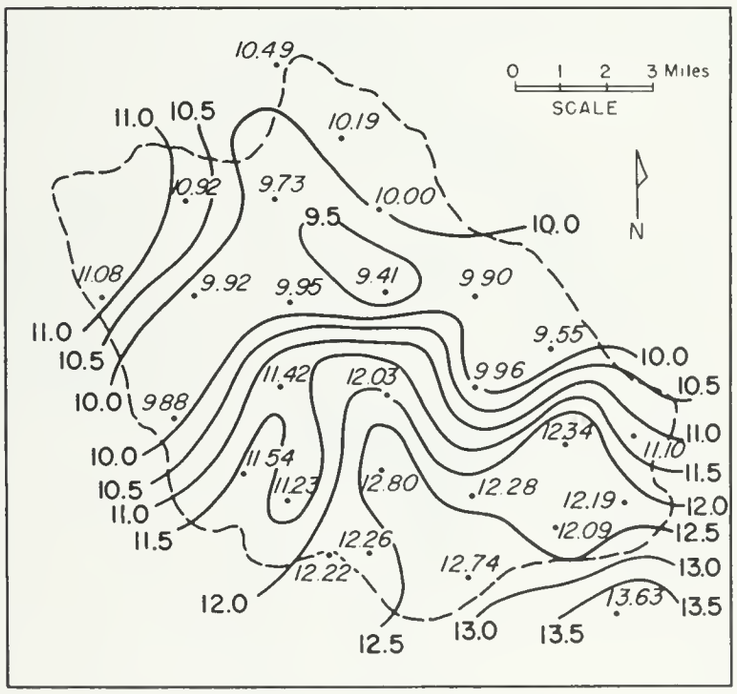
A. STORM RAINFALL, JULY 1, 1953
($\overline{P}$=.52, CV=.30)



B. WEEKLY RAINFALL, JULY 1-7, 1953
($\overline{P}$=3.57, CV=.11)



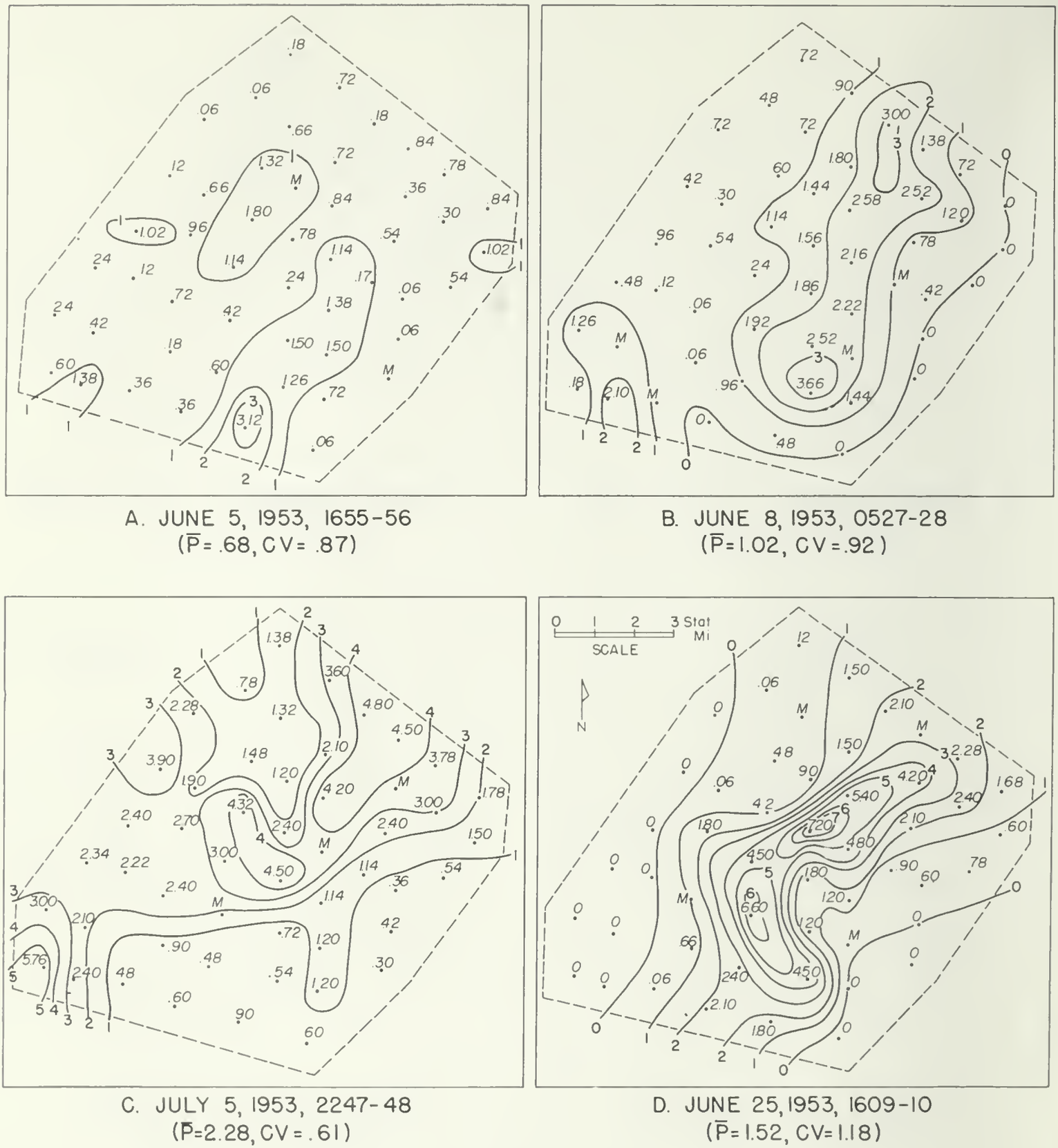
C. JULY 1953 RAINFALL
($\overline{P}$=6.36, CV=.15)



D. JUNE-AUG. 1953 RAINFALL
($\overline{P}$=11.03, CV=.10)

FIGURE 10 EXAMPLES OF RAINFALL VARIABILITY

The tendency for the relative variability to decrease with increasing mean rainfall and with the length of observation period is illustrated in Figure 10. This figure shows the isohyetal maps for storm, weekly, monthly, and seasonal (June-August) rainfall during the summer of 1953 on the Panther Creek network. In the illustration, $\overline{P}$ is the mean rainfall and CV is the coefficient of variation.



A. JUNE 5, 1953, 1655-56
($\bar{P}$ = .68, CV = .87)

B. JUNE 8, 1953, 0527-28
($\bar{P}$ = 1.02, CV = .92)

C. JULY 5, 1953, 2247-48
($\bar{P}$ = 2.28, CV = .61)

D. JUNE 25, 1953, 1609-10
($\bar{P}$ = 1.52, CV = 1.18)

FIGURE 11 ONE-MINUTE MEAN RATES

Figure 11 shows some examples of relative variability in thunderstorms when mean rainfall is calculated on a very short time basis. The four isohyetal maps are for one-minute periods measured on the Goose Creek network during 1953, using special recording rain gages (see previous section and Figure 4). Isohyetal values in Figure 11 represent one-minute amounts expressed in rainfall rate in inches per hour. $\bar{P}$ represents the mean rainfall rate and CV the coefficient of variation. Since the rainfall values on the maps represent nearly instantaneous values, the isohyetal patterns are approximately representative of the instantaneous pattern of rainfall within thunderstorms. The great variability existing in the internal structure of thunderstorms is apparent and helps explain the great variability observed in total storm rainfall in thunderstorms and the scatter in the coefficient of variation shown in Figure 9.

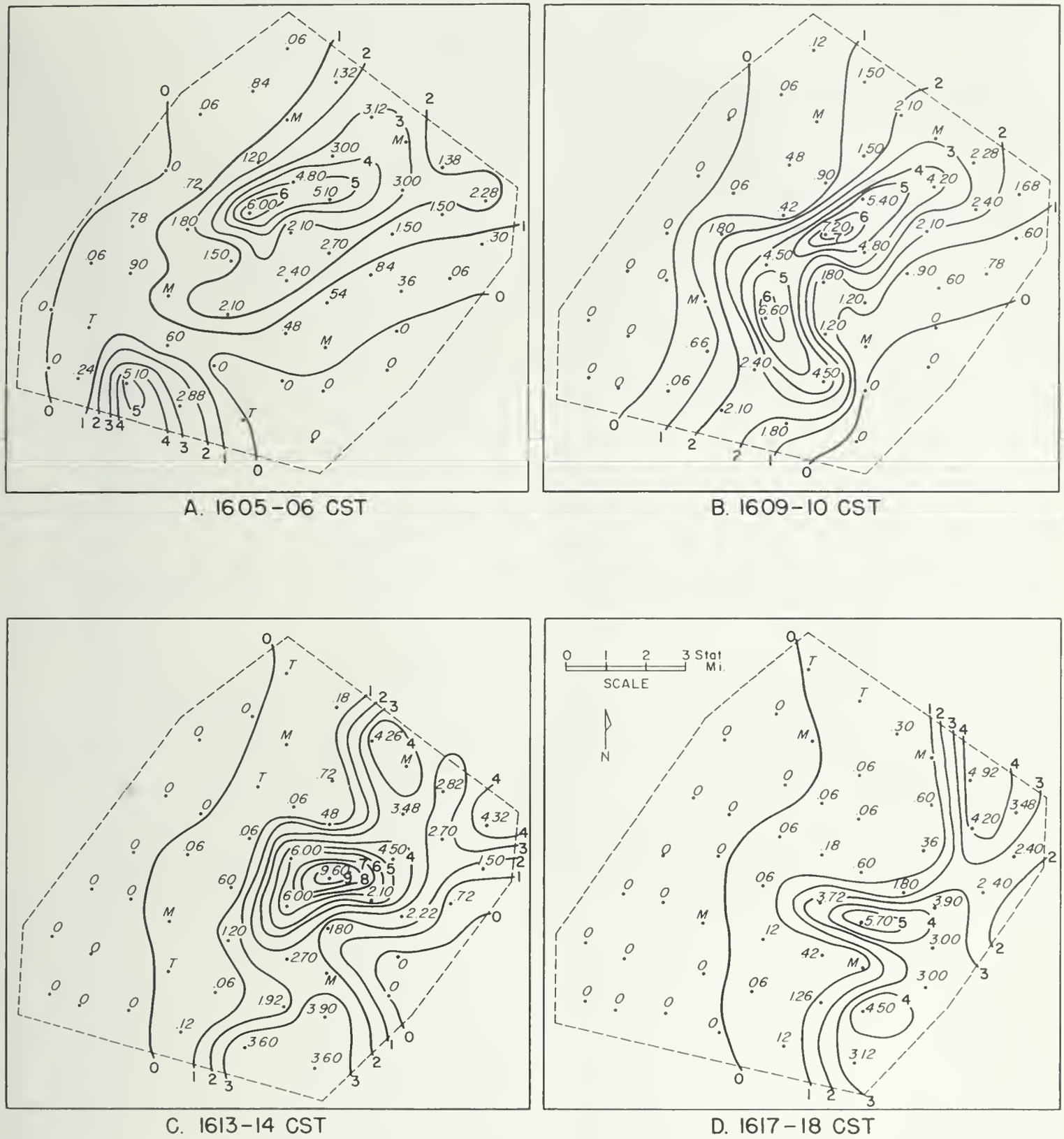


FIGURE 12 TIME VARIATIONS IN RAINFALL PATTERN

Figure 12 is a series of one-minute isohyetal maps spaced four minutes apart in an intense thunderstorm and illustrates the rapid changes which may take place in the internal structure of thunder-storms.

The effects of storm rainfall variability will be discussed in the section on area-depth curves, where its application to hydrologic analysis and design will be shown.

Effect of Gage Density on Maximum Recorded Point Rainfall

The peak rainfall occurring within basin storms is a useful factor in hydrologic analysis. Obviously, the maximum recorded rainfall in shower-type storms is closely related to the gage density on a given basin.

Using rainfall data collected from 70 storms on the Panther Creek Basin during 1948-50, a study was made of the effect of gage density on the observed maximum point rainfall within an area. To study this effect, point rainfall recorded at the most centrally-located station in the 100-square mile basin was compared with the maximum point rainfall obtained by increasing the network progressively to 3, 5, 10, and 20 gages. These networks, corresponding to gage densities of 33, 20, 10, and 5 square miles per gage, were chosen to give the most uniform distribution of gages possible.

For each of the four gage densities in each storm, the ration of network maximum to single gage amount was determined from the rain-gage charts. These data were then grouped, based upon class intervals for single gage rainfall of 0.01 - 0.20, 0.21 - 0.50, 0.51 - 1.00 and 1.01 - 2.00 inches. The grouped data were then used to obtain an empirical relation between network maximum rainfall, single gage rainfall, and gage density. The empirical equation obtained is

$$\frac{R_m}{R_s} = 2.5\ G^{-.20} \qquad (2)$$

where R_m is the maximum recorded point rainfall (in.), R_s is the central gage amount (in.) and G is the gage density (sq. mi./gage). The developed relation is illustrated in Figure 13.

Reference to Figure 13 shows the expected trend for the maximum recorded point rainfall to increase significantly as the gage density decreases (number of gages increases). The average ratio of maximum recorded to central gage rainfall was found to increase from 1.15 at a gage density of 50 square miles per gage to 1.38, 1.58 and 1.83 at gage densities of 20, 10 and 5 square miles per gage, respectively.

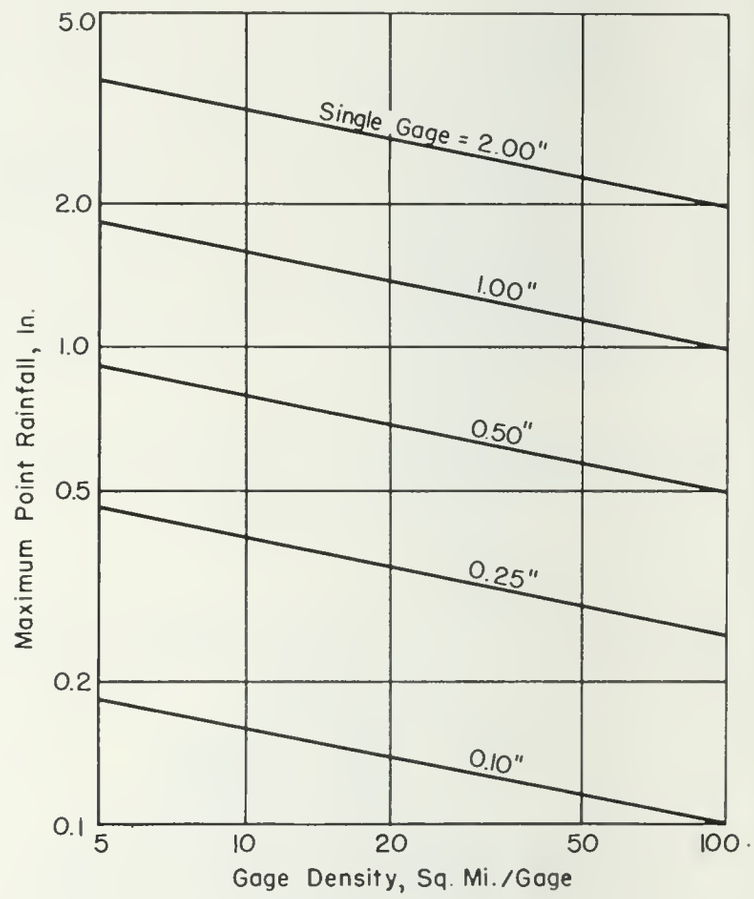


FIGURE 13 EFFECT OF GAGE DENSITY ON MAXIMUM RECORDED POINT RAINFALL

FREQUENCY OF POINT AND AREAL MEAN RAINFALL RATES

Introduction

During 1952-53, one minute rainfall totals for a number of storms were tabulated in conjunction with a project concerned with the investigation of the utility of radar for quantitative rainfall measurements.[2] Use of special recording rain gages with 12.6 -inch diameter orifices and six-hour charts enabled recording of one-minute amounts over an area of 100 square miles. Information on the distribution of rainfall rates within storms should be of interest to hydrologists, especially to those concerned with sedimentation. Areal distributions of one-minute rates, which closely approximate the actual pattern of rainfall rates within storms, are pertinent in studying the physical structure of storms. Information on short-period rates is also valuable in evaluating radar for quantitative precipitation measurements and other meteorological uses, since precipitation attenuation is proportional to the rainfall rates within a storm, and attenuation presents an outstanding problem in applying radar to the measurement of rainfall intensity.[3] Consequently, an analysis was made of the frequency of occurrence of point and areal mean rainfall rates. The available data were restricted to shower-type rainfall occurring during the thunderstorm season from late spring to early fall.

Data Used in Analysis

One-minute rainfall amounts from 19 storms during 1952-53, tabulated in conjunction with the radar-rainfall project mentioned previously, were used in the study. The one-minute amounts were obtained from 50 recording gages located on the 100-square mile Goose Creek Network(Fig.4).* The distribution of gages on the network was approximately uniform, the uniformity being limited only by existing road systems and suitable exposures. The network terrain is relatively flat with no distinct topographic features to influence storm behavior.

Careful attention was given to synchronizing the gages so that one-minute rainfall patterns and areal means could be reliably represented. All clocks were checked for accurate timing before installation each spring. Before proceeding to the network to change charts, observers synchronized their watches with a master clock, which was checked daily with time signals from radio station WWV, the National Bureau of Standards Station at Washington, D.C. When installing and removing charts from the drum, the observer made a verticle time check mark with the pen on the chart and entered the exact "on" or "off" time from his watch. Charts were changed after each storm and twice a week during periods of no rain to minimize any clock timing errors. When a rain-gage record indicated the gage clock had gained or lost a small amount of time, the error was prorated over the period of record. As a further check, after isohyetal maps were plotted for each minute of a storm, gage charts were reexamined if doubtful values appeared at any stations on a one-minute map.

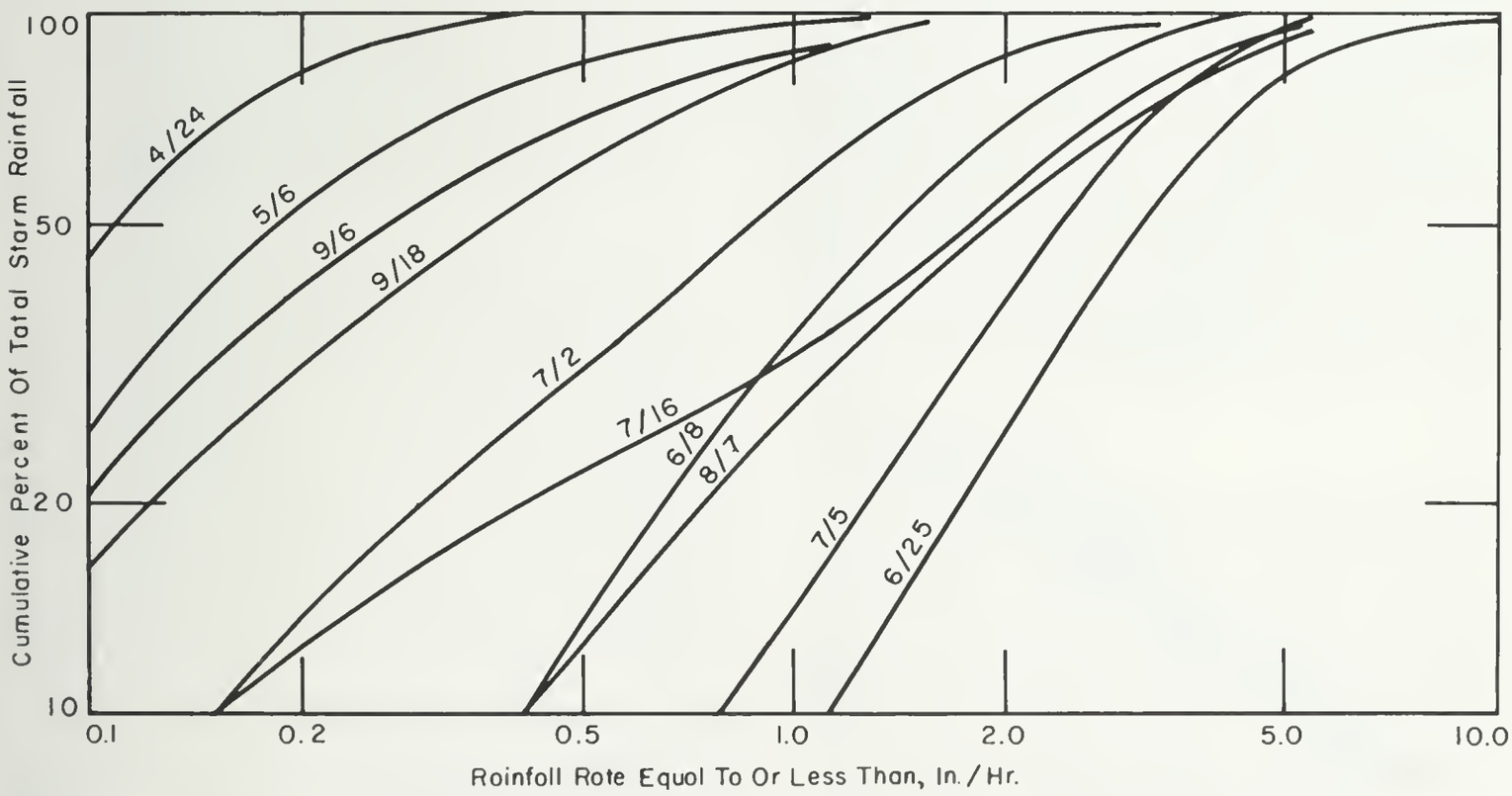


FIGURE 14 DISTRIBUTION OF POINT RAINFALL RATES IN INDIVIDUAL STORMS

*Operation of the network and data reduction were sponsored in part by the Signal Corps Engineering Laboratories, Belmar, New Jersey under contract DA-36-039 SC-42446

Frequency of Point Rainfall Rates

The frequency with which various point rainfall rates occurred in each storm was calculated first, using the data from all 50 gages. Figure 14 shows cumulative frequency curves for the ten 1953 storms. The graph shows cumulative percentages plotted against rainfall rate in inches per hour, based upon one-minute amounts from all 50 gages. For example, in the storm for July 2, 50 per cent of the rainfall occurred at rates exceeding 0.90 inch per hour, while for the storm of June 25, 50 per cent of the total rainfall occurred at rates exceeding 3.10 inches per hour.

Examination of the 19 storms indicated that the group should be approximately representative of conditions experienced during the warm season in the Midwest. Consequently, the data were combined to determine the frequency of point rainfall rates under average conditions during a warm season. Results of this analysis are shown in columns 1 and 2 of Table 4. It will be noted that on the average, based on one-minute rates in the 19-storm sample, 50 per cent of the warm season rainfall occurs at rates exceeding 1.45 inches per hour. However, the median rate in individual storms in the 19-storm sample was highly variable, ranging from 0.10 to 3.10 inches per hour.

For comparison purposes, the per cent of total storm rainfall occurring at various rainfall rates when these rates are based on 5-, 10-, 15-, and 30-minute summation periods was determined. Obviously, the averaging process suppresses details of the high rates occurring during the averaging period. Most studies in the past, such as those of Yarnell[4] and the U.S. Weather Bureau[5] on which engineering design has been based, were calculated on the basis of periods from five minutes upward. Hydrologists have found that rainfall rates for periods less than 10 to 15 minutes seldom have practical significance with respect to runoff from a drainage area. However, as mentioned previously, shorter period rates have application in sedimentation studies, in gaining an understanding of the physical structure of storms, and in the evaluation of radar for meteorological purposes. In the present study, an attempt was made to ascertain the effect of the averaging process on the observed frequency of rainfall rates.

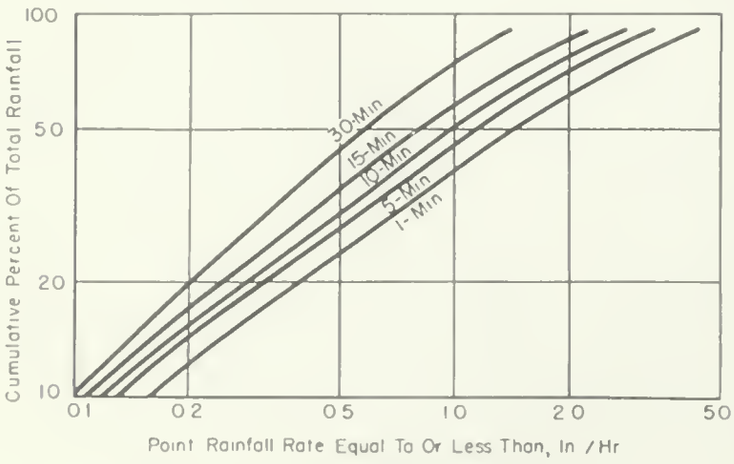


FIGURE 15 DISTRIBUTION OF POINT RAINFALL RATES DURING AVERAGE SEASON

TABLE 4

FREQUENCY OF POINT RAINFALL RATES CALCULATED FROM DIFFERENT BASE PERIODS

Rainfall Rate (in/hr) Equaled or Exceeded For Several Base Periods (Min.)

Cumulative Per Cent	1	5	10	15	30
Max.	13.80	7.50	5.72	4.20	2.45
5	5.35	4.10	3.25	2.64	1.62
10	4.20	3.35	2.80	2.34	1.41
20	3.25	2.58	2.13	1.93	1.15
30	2.45	2.02	1.75	1.42	0.88
40	1.90	1.52	1.34	1.07	0.69
50	1.45	1.16	0.97	0.82	0.56
60	1.05	0.83	0.70	0.62	0.44
70	0.68	0.54	0.46	0.41	0.33
80	0.37	0.28	0.25	0.23	0.21
90	0.15	0.12	0.11	0.11	0.10
95	0.08	0.07	0.07	0.07	0.06

Per Cent of 1-Min. Rate for Several Periods (Min.)

Cumulative Per Cent	5	10	15	30
Max.	54	41	30	18
5	77	61	49	30
10	80	67	56	34
20	79	66	59	35
30	82	71	58	36
40	80	71	56	36
50	80	67	57	39
60	79	67	59	42
70	79	68	60	48
80	76	68	62	57
90	80	73	73	67
95	87	87	87	75

Results are illustrated in Figure 15 and Table 4. In Table 4 (upper part) it is seen that when one-minute totals are used, 50 per cent of the rain was found to occur at rates exceeding 1.45 inches per hour. When a five-minute base period was used, the 50 per cent value dropped to 1.16 inches per hour and then to 0.97 inch per hour for a 10-minute base period, 0.82 inch per hour for a 15-minute base period, and 0.56 inch per hour for a 30-minute base period. Also, in Table 4 note the rapid decrease in the maximum observed rainfall rate when this rate is based on various averaging periods. The averaging effect is further illustrated in the lower part of Table 4, where the longer period values have been expressed as percentages of the one-minute values.

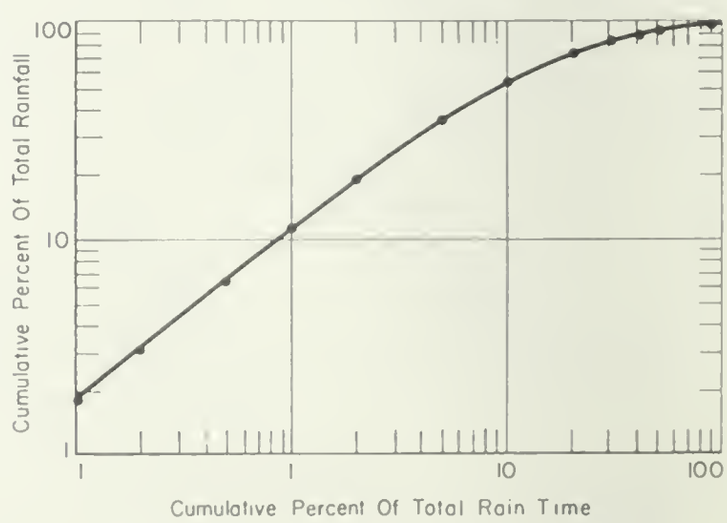


FIGURE 16 TIME DISTRIBUTION OF POINT RAINFALL

The time distribution of point rainfall was studied next, combining data from all 19 storms. Results of this analysis are shown in Figure 16 where per cent of total time of rainfall in the 19 storms has been plotted against per cent of total rainfall in these storms. From this curve it is seen that 10 per cent of the season rainfall, based on the 19-storm sample, occurs in approximately one per cent of the time it is raining, 50 per cent of the rain occurs during about 9 per cent of the time that rain is falling, and 90 per cent of the total rain occurs in 50 per cent of the time that it is raining. Thus, it is apparent that warm season rainfall tends to occur in strong bursts within a storm period; that is, most of the rain tends to occur in a small portion of the storm.

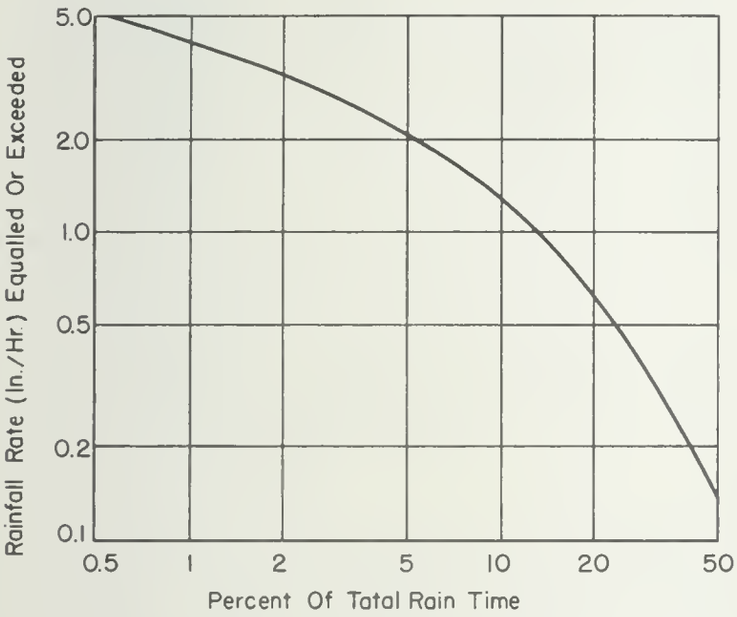


FIGURE 17 POINT RAINFALL TIME-RATE RELATION

Figure 17 shows a time-rate relation for point rainfall derived from the combined data for the 19 storms. This graph further emphasizes the burst nature of shower-type rainfall. Reference to Figure 17 shows that rates equalling or exceeding 1.45 inches per hour, which accounted for 50 per cent of the total rainfall according to Figure 15, occurred only 9 per cent of the time it was raining. Similarly, rates exceeding 4.25 inches per hour occurred only one per cent of the time rain was falling, while rates exceeding 0.14 inch per hour were recorded during 50 per cent of the time it was raining.

Frequency of Areal Mean Rainfall Rates

The foregoing discussion has been concerned with the frequency of point rainfall rates. Often, hydrologists are more concerned with areal than with point relationships. Consequently, data from the 50-gage network on the 100-square mile area were further analyzed to obtain areal frequency relationships. For this purpose, the 100-square mile network was subdivided to give areas of 10, 25, and 50 square miles for comparisons with areal mean storm rainfall relations on the 100-square mile network. One-minute areal mean rates were calculated for each minute in each storm for each of the areas mentioned above.

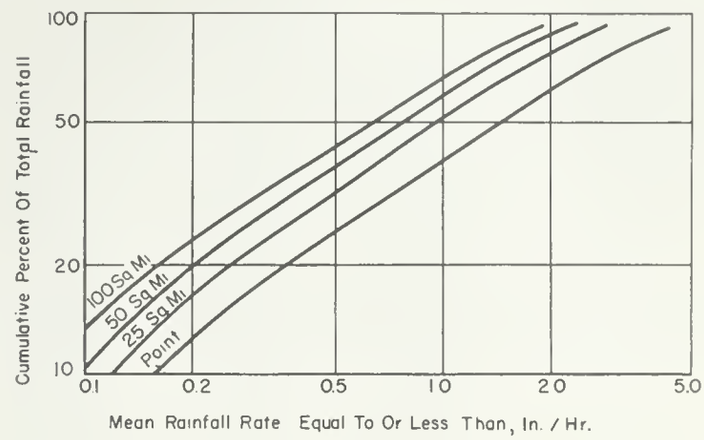


FIGURE 18 DISTRIBUTION OF AREAL MEAN RAINFALL RATES

All 19 storms were combined to obtain average relationships for the various areas. The results are summarized in Tables 5 and 6 and Figure 18. Reference to Table 5 shows the expected trend. For example, 50 per cent of the point rainfall was found to occur at a rate exceeding 1.45 inches per hour, while 50 per cent of the areal mean rainfall occurred at rates of 1.02, 0.96, 0.78 and 0.66 inches per hour, respectively, for the 10-, 25-, 50-, and 100-square mile areas. Table 6 illustrates the variation in the time distribution of rainfall rates as the area is increased. For example, 50 per cent of the point rainfall was found to occur during 9 per cent of the time it was raining. At 25 square miles, 50 per cent of the areal mean rainfall occurred in 11 per cent of the time, while for 100 square miles 50 per cent occurred in 12 per cent of the rain time. This table shows a tendency for the time distribution to change only very slowly with increasing area.

Discussion of Results

Although the results in this study are based upon a relatively small sample, the variety of storms incorporated in the 19-storm sample should make the results approximately representative of average warm season conditions. It would have been desirable to have included more storms; however, the great amount of hand tabulation required did not permit it. It is hoped that the results will prove useful in evaluating the actual distribution of rainfall rates in warm season rainfall in Illinois and the Midwest. At the present time, machine processing of the rain-gage charts is being investigated, and if it is feasible, the present investigation can be greatly expanded since a large amount of unprocessed data are available which were collected from 1951-54.

TABLE 5

FREQUENCY OF AREAL MEAN RAINFALL RATES

Cumulative Per Cent of Rainfall	Rainfall Rate (in/hr) Equaled or Exceeded for Given Area (Mi.^{2})					Per Cent of Point Rate			
	Point	10	25	50	100	10	25	50	100
Max.	13.80	4.08	4.02	3.18	2.88	30	29	23	21
5	5.50	3.48	2.82	2.52	2.40	63	51	46	44
10	4.20	3.18	2.55	2.22	1.86	76	61	53	44
20	3.25	2.28	2.22	1.74	1.35	70	68	54	42
30	2.45	1.80	1.77	1.32	1.11	73	72	54	45
40	1.90	1.26	1.26	1.02	0.90	66	66	54	47
50	1.45	1.02	0.96	0.78	0.66	70	66	54	46
60	1.05	0.72	0.72	0.54	0.48	69	69	51	46
70	0.68	0.48	0.48	0.36	0.28	71	71	53	41
80	0.37	0.25	0.26	0.18	0.16	68	70	49	43
90	0.15	0.11	0.12	0.10	0.08	73	80	67	53
95	0.08	0.07	0.07	0.05	0.04	87	87	62	50

TABLE 6

TIME DISTRIBUTION OF AREAL MEAN RAINFALL RATES

Area (Mi.)	Per Cent of Total Rain Time Accounting for Given Per Cent of Total Rainfall				
	10	25	50	75	90
Point	1	3	9	25	48
10	1	3	10	26	51
25	1	4	11	27	53
50	1	4	12	28	55
100	1	4	12	30	58

AREA-DEPTH RELATIONS

Introduction

For engineering design purposes, a knowledge of detailed area-depth relations for small areas is often essential. Few detailed data for areal units under 300 square miles have been published. Rain-gage networks of sufficient density to define area-depth relations accurately from isohyetal maps for such small areas are uncommon. While publications of the Miami Conservancy District [6] and the U.S. Corps of Engineers [7] include data on small watersheds, the data are based on relatively sparse spacing of rain gages. The concentrated networks employed in this study were designed for collecting more precise data.

To obtain detailed information on the characteristics of the area-depth curve for small watersheds, thunderstorm and rainshower data collected on four densely-gaged networks in central Illinois have been analyzed. These three networks consist of the 280-square mile El Paso network (Fig.2), the 100-square mile Panther Creek (Fig. 3) and Goose Creek (Fig. 4) networks, and the Boneyard Creek network of 5.5 square miles (Fig. 6). The necessity for concentrated networks, which can be used to obtain detailed isohyetal patterns and thereby minimize interpolation errors in the construction of area-depth curves, is illustrated by the series of isohyetal maps in Figure 19. These maps were drawn for the same storm on the 100-square mile Panther Creek watershed using the various gage densities indicated.

Analysis and Results

Most previous investigations of area-depth relations have been concerned with relatively large watersheds, employing less dense networks than those utilized in this study. It has been observed that area-depth curves derived from isohyetal maps sometimes approach straight lines or flat curves when the logarithm of area is plotted against average rainfall. [8] This often results from a liberal envelopment of points. Curved lines were obtained with the Illinois area-depth data in most cases by plotting the data on semi-logarithmic paper with depth on the linear scale and area on the logarithmic scale.

Since a straight-line representation is desirable, several hypotheses were investigated in an effort to obtain an easily computed straight-line relation. Within the limits tested for the four small areas, it was found that the data conformed closely to the general equation [9]

$$Y = a + b X^{1/2} \qquad (1)$$

where Y is average rainfall depth in inches, X is the area enveloped in square miles, and a and b are constants representing maximum storm point rainfall and mean rainfall gradient, respectively.

The area-depth curve is a two-dimensional representation of rainfall distribution over area, in which values of average rainfall within an isohyet are plotted against the area enveloped. The area-depth curve is constructed as though the highest value of rainfall occurred at one point, and lower values appeared (in an isohyetal representation) roughly concentrically around the higher values.

Chow [10] has pointed out that Equation (1) indicates that the three-dimensional representation of the rainfall isohyets, which he calls the "storm pile," of thunderstorms or rainshowers is very close to the form of a cone. He indicated that additional terms of higher orders of X must be considered when the straight-line relationship of the depth plotted against the square root of the area does not apply to storms covering larger areas, that is more than 300 square miles.

Within the "storm pile," point rainfall is also linearly related to the distance from the center of the storm as represented by the following equation

$$y = A + Br \qquad (2)$$

where y is the point rainfall at a distance, r, from the center of the storm, and for a given storm, A and B are constants, representing maximum point rainfall and point rainfall gradient. Analysis [11] has shown that A = a in Equation (1) and b = 0.376B, so that

$$Y = A + 0.376B\, X^{1/2} \qquad (3)$$

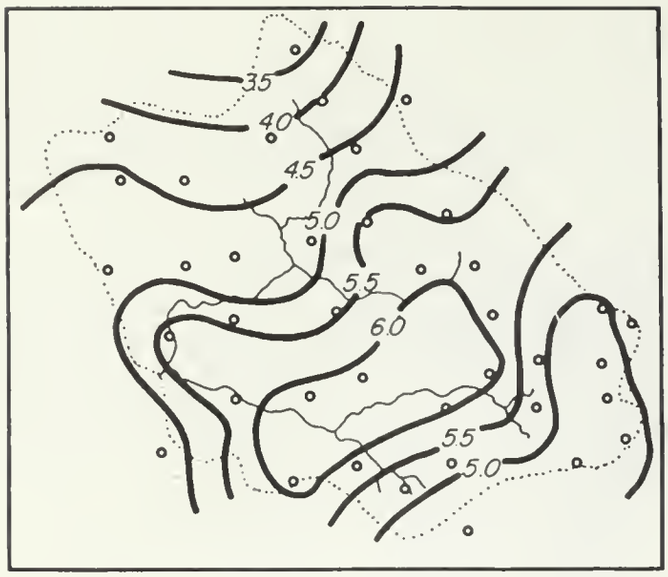
Equations (1) to (3) can be used to construct hypothetical area-depth curves for design purposes on small basins, when the maximum point rainfall of the storm center can be obtained from frequency data, such as those obtained by Yarnell, [4] provided that the rainfall gradient or mean rainfall on the basin can be estimated from a local rain-gage network. Development of a relationship for rainfall gradient will be discussed later.

Equation (1) was tested on storms ranging from one to 24 hours duration on the four areas. Only storms in which the areal mean rainfall equaled or exceeded 0.50 inch were used. These included 126 storms on the two 100-square mile networks, 72 on the 5.5-square mile area, and 19 on the 280-square mile network. Results for the heaviest storms in each area are illustrated in Figures 20-22.

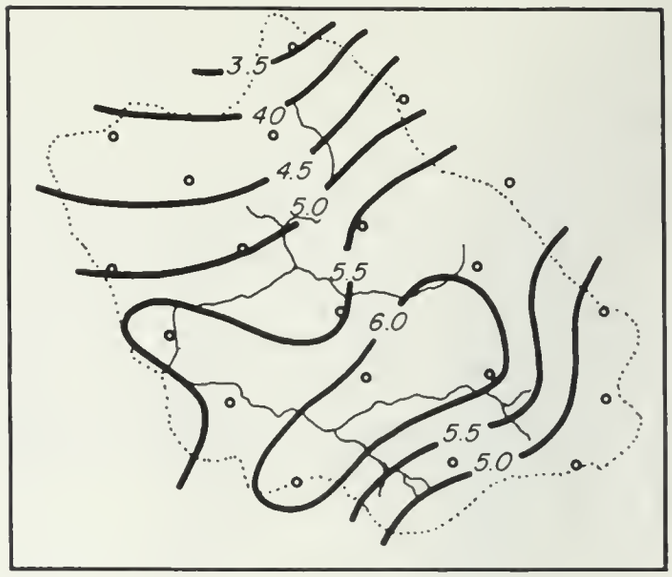
To make the area-depth relation developed in Equation (1) more applicable for engineering design purposes, data from the 5.5- and 100-square mile areas were analyzed further to determine the relationship between the curve slope (rainfall gradient) and other storm parameters. Analysis was not accomplished for the 280-square mile area due to the limited data available. Examination of the data for the two smaller areas indicated that the best fit would be obtained with a relationship of the form

$$b = K\, P^{l}\, T^{m} \qquad (4)$$

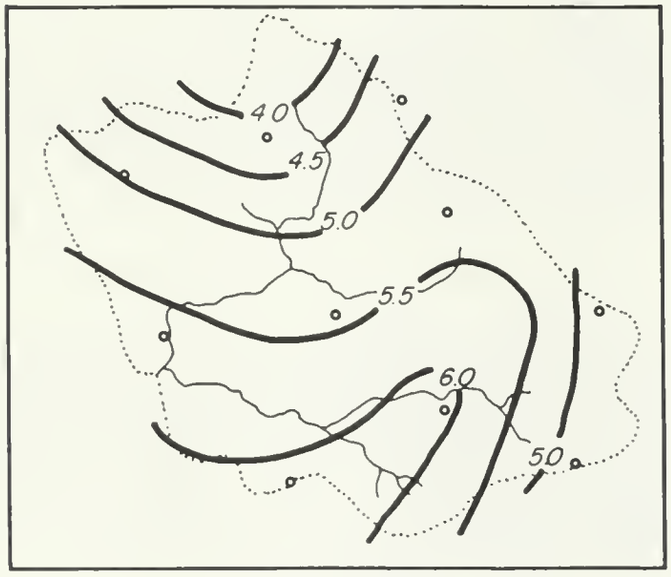
where b is the mean rainfall gradient in Equation (1), P is areal mean rainfall, T is storm duration, and K, l, and m are constants.



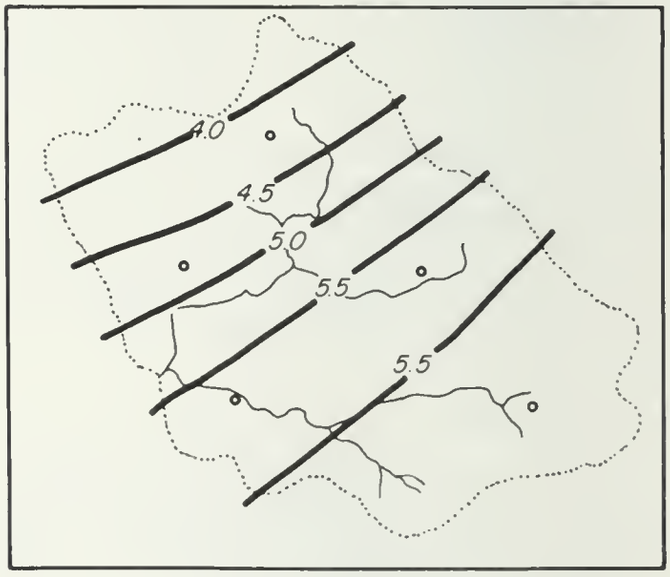
2.4 sq mi per gage



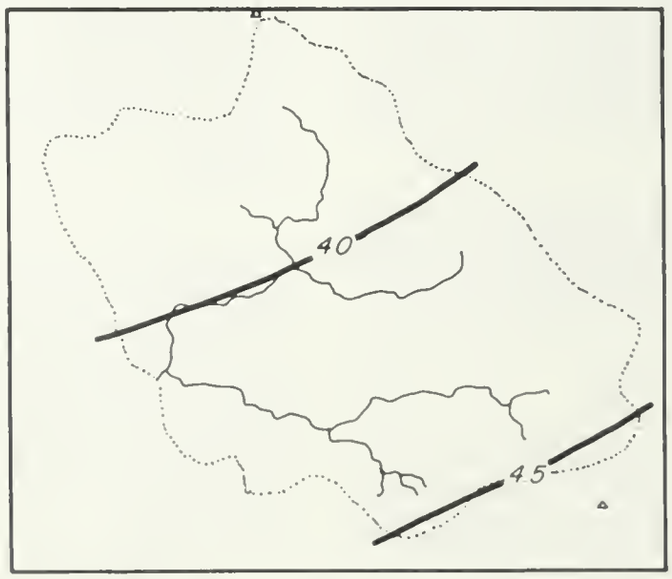
4 75 sq. mi. per gage



9.5 sq. mi per gage



19.5 sq. mi per gage



U S Weather Bureau Climatological Network, 225 sq mi per gage

FIGURE 19 EFFECT OF GAGE DENSITY ON ISOHYETAL PATTERN

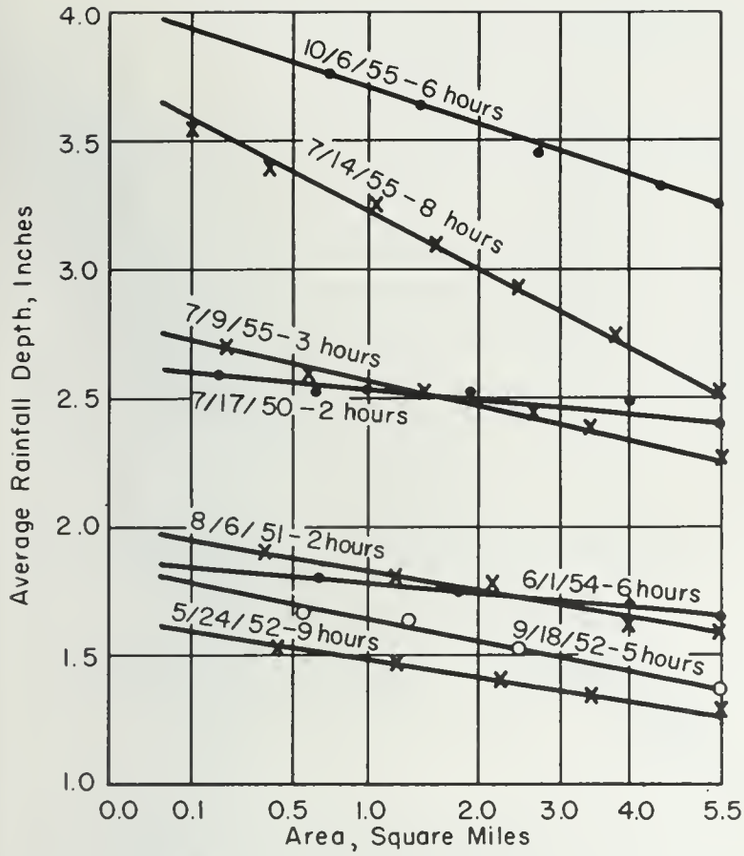


FIGURE 20 AREA-DEPTH RELATIONS FOR HEAVIEST STORMS, 5.5 SQUARE MILE AREA

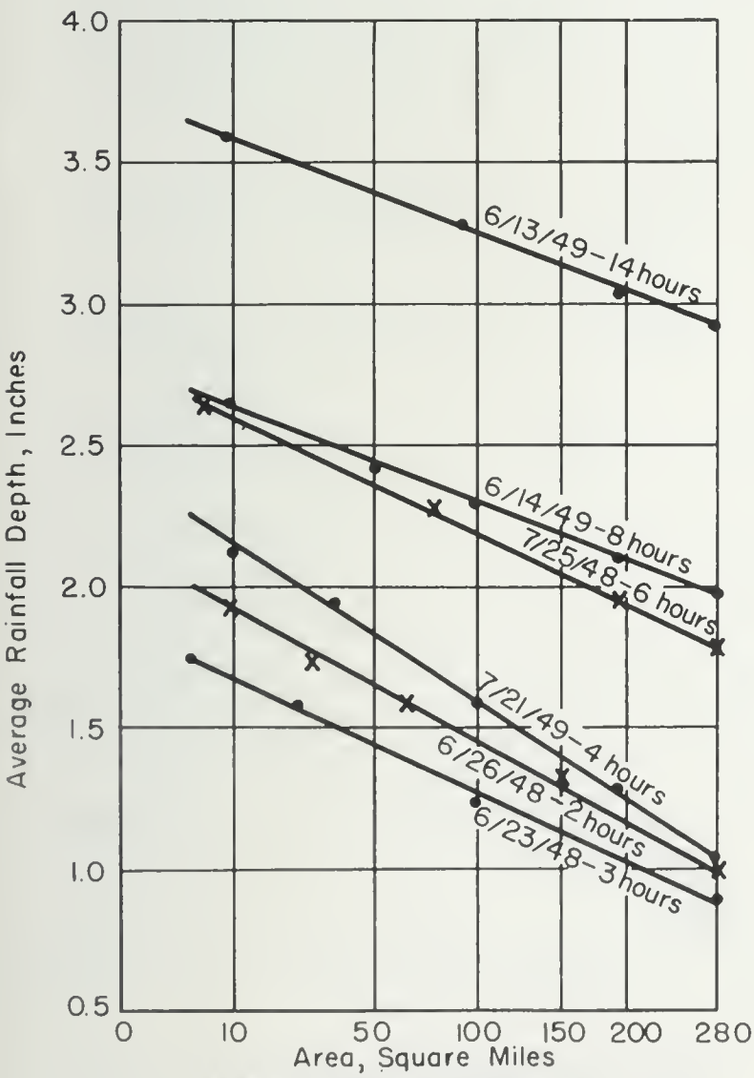


FIGURE 22 AREA-DEPTH RELATIONS FOR HEAVIEST STORMS, 280 SQUARE MILE AREA

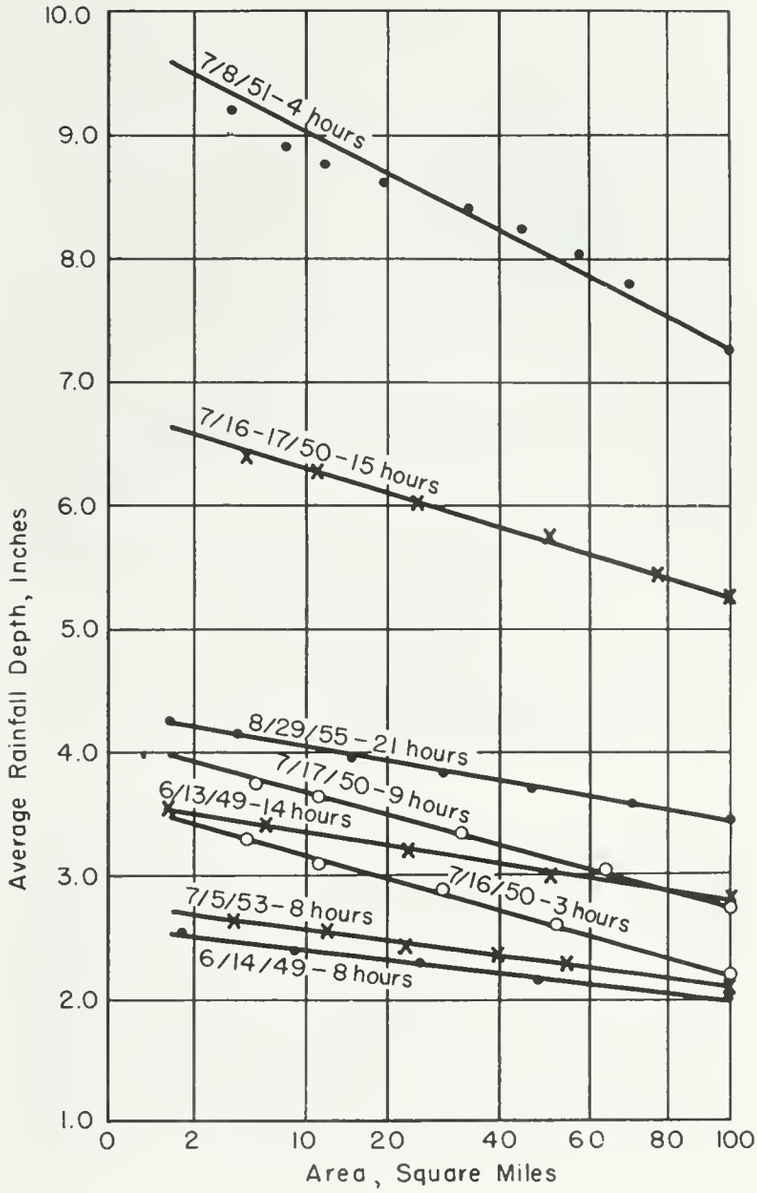


FIGURE 21 AREA-DEPTH RELATIONS FOR HEAVIEST STORMS, 100 SQUARE MILE AREA

Results of this study yielded Equations (5) and (6) for the 100- and 5.5-square mile basins, respectively,

$$b = 0.094\ P^{.83}\ T^{-.28} \qquad (5)$$

$$b = 0.169\ P^{.89}\ T^{-.15} \qquad (6)$$

As expected, the data indicated considerable variability in the rainfall gradient among storms of similar mean rainfall and duration. Obviously, there are other factors in addition to storm size and storm duration, that are not easily reduced to mathematical expression, which affect the calculated rainfall gradient within storms. For example, the area-depth curve for a given basin will be influenced by the location of the storm core with respect to the basin, movement of the storm with respect to the axis of the basin, and the internal structure of the storm. A multiple correlation coefficient of 0.65 was obtained from Equation (5) and a coefficient of 0.55 for Equation (6). These relatively low coefficients reflect the high variance of the data.

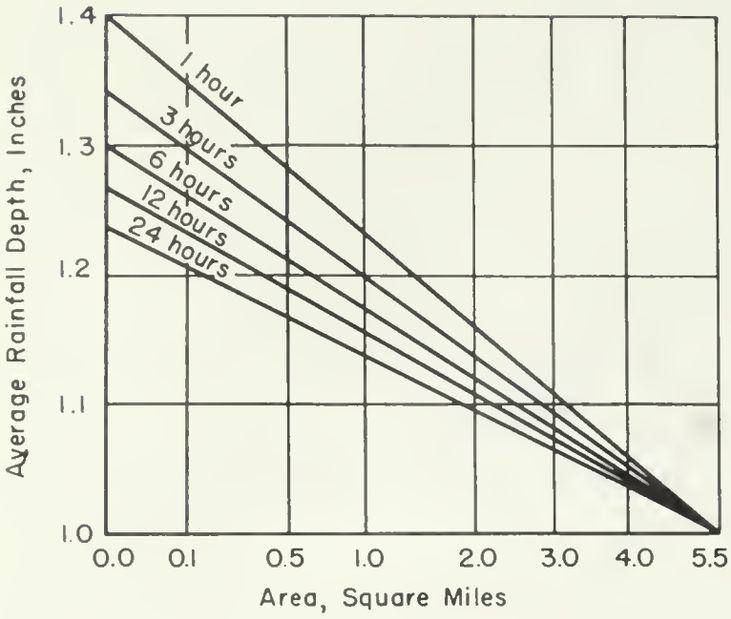


FIGURE 23 AVERAGE AREA-DEPTH RELATIONS, 5.5 SQUARE MILE AREA, MEAN RAINFALL = 1.00 INCH

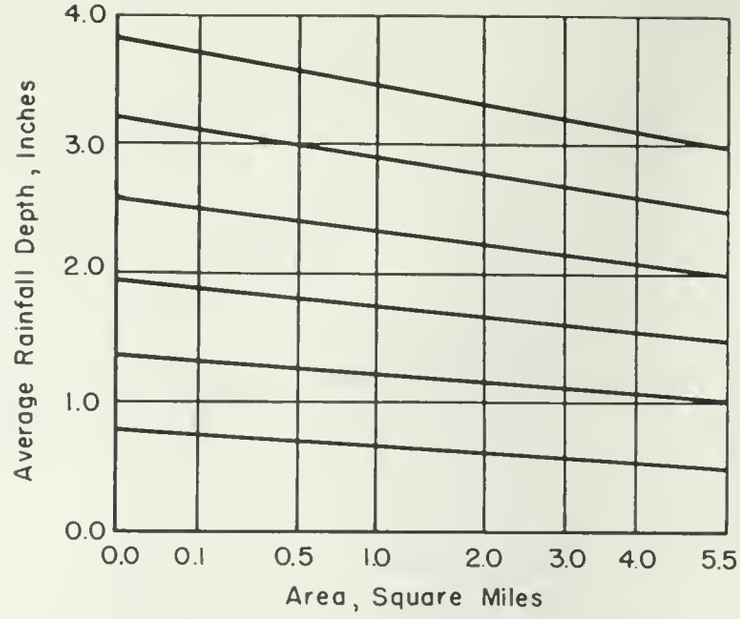


FIGURE 25 AVERAGE AREA-DEPTH RELATIONS, 5.5 SQUARE MILE AREA, 6-HOUR STORM DURATION

Applying the relations for the curve slope or rainfall gradient in Equations (5) and (6) to Equation (1), average area-depth relations were developed for the two areas and are illustrated in Figures 23 to 26. Figures 23 and 24 show average area-depth relations for various storm durations based on an areal mean rainfall of one inch. Figures 25 and 26 show average area-depth relations for storms of various magnitude (mean rainfall) for a storm duration of six hours. Obviously, the developed equations can be used in calculations for other storm sizes and durations.

All data for both areas are for storm durations up to 24 hours; consequently, their curves should not be used for longer period storms. It should also be pointed out that the mean rainfalls tested for the 5.5-square mile area did not exceed 3.5 inches. Except for two storms, all storms observed on the 100-square mile networks had less than 5-inch means. Consequently, upper limits of 3-inch and 4-inch means have been used in Figures 25 and 26.

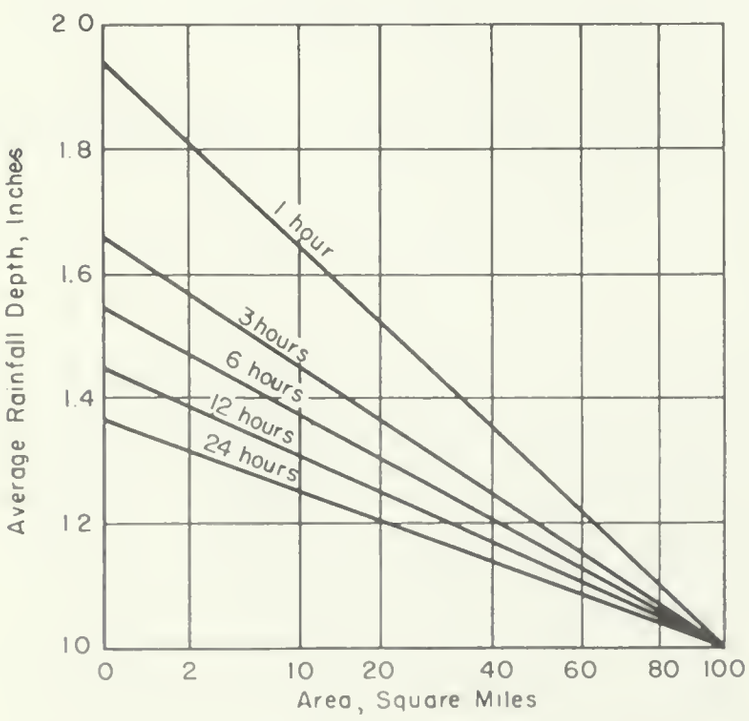


FIGURE 24 AVERAGE AREA-DEPTH RELATIONS, 100 SQUARE MILE AREA, MEAN RAINFALL = 1.00 INCH

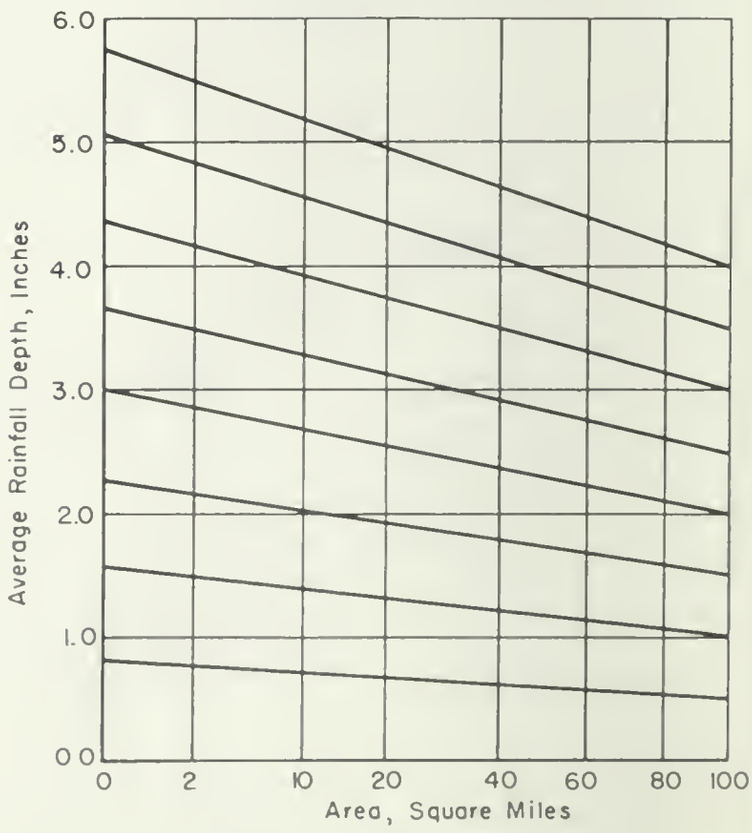


FIGURE 26 AVERAGE AREA-DEPTH RELATIONS, 100 SQUARE MILE AREA, 6-HOUR STORM DURATION

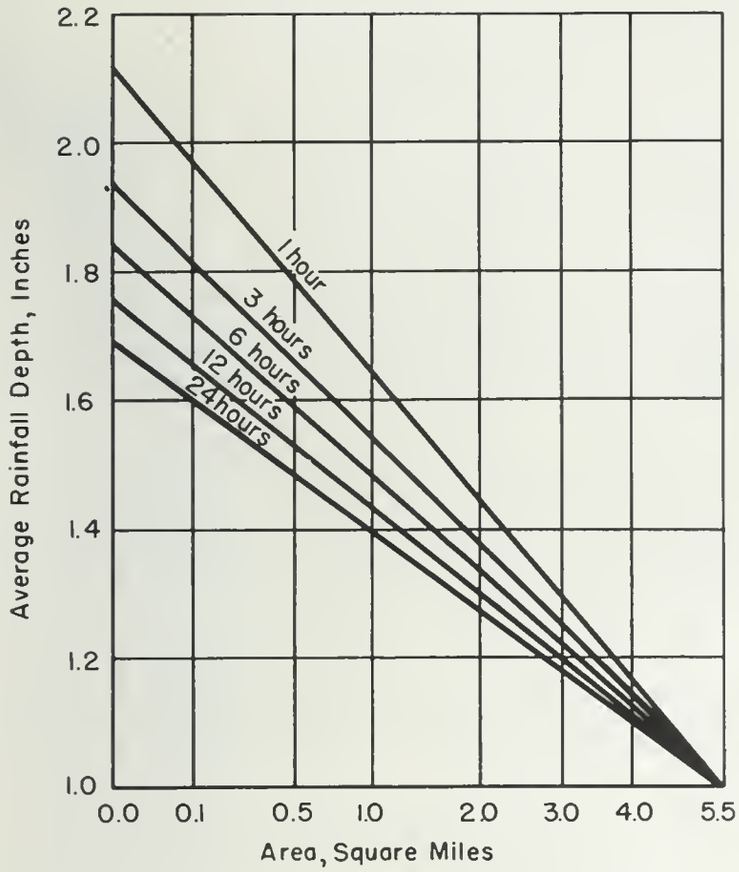


FIGURE 27 EXTREME AREA-DEPTH RELATIONS, 5.5 SQUARE MILE AREA, MEAN RAINFALL = 1.00 INCH

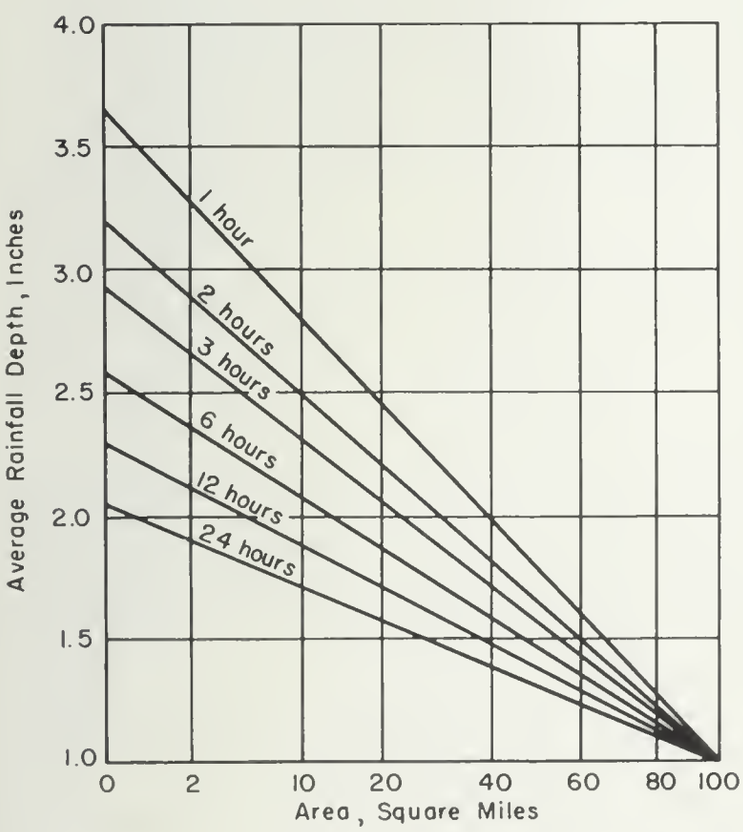


FIGURE 28 EXTREME AREA-DEPTH RELATIONS, 100 SQUARE MILE AREA, MEAN RAINFALL = 1.00 INCH

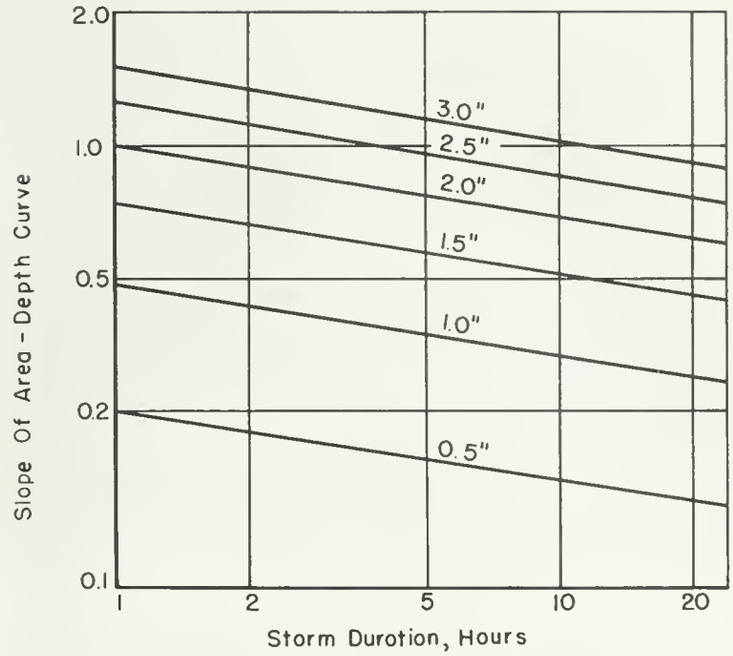


FIGURE 29 EXTREME AREA-DEPTH CURVE SLOPES, 5.5 SQUARE MILE AREA, SELECTED AREAL MEAN RAINFALLS

The curves in Figures 23 to 26, of course, are for average values of rainfall gradient (curve slope). For both Equations (5) and (6), 95 per cent confidence bands were determined. The upper 95 per cent confidence band in both cases has been used in Figures 27 and 28 to show extreme area-depth relations for the two basins with a mean rainfall of one inch. Figures 29 and 30 show extreme curve slopes for durations of 1 to 24 hours and for various mean rainfalls. These figures may be used in conjunction with Equation (1) to obtain

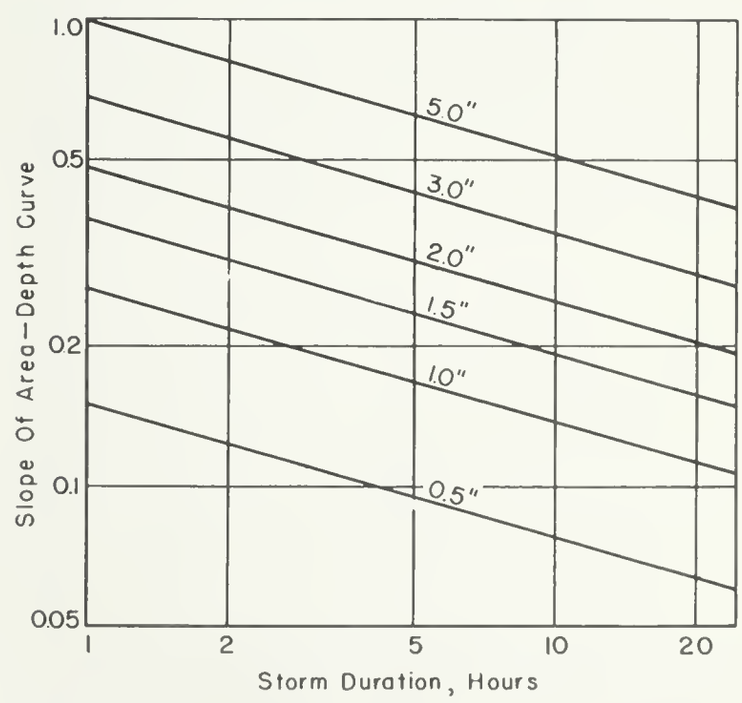


FIGURE 30 EXTREME AREA-DEPTH CURVE SLOPES, 100 SQUARE MILE AREA, SELECTED AREAL MEAN RAINFALLS

extreme area-depth curves under various conditions. Whether average or extreme values should be used depends upon the purpose for which area-depth relations are to be utilized. Again, the large differences between the average curves in Figures 23-24 and the extreme curves in Figures 27-28 result from the large variability in rainfall gradients among storms of similar size and duration.

In Equations (5) and (6) for the slope of area-depth curves, it is interesting to note the relative weight of the two parameters, mean rainfall and storm duration. The relative importance of mean rainfall is greater in the equations for both basins as illustrated by the exponents on P and T. Results indicate that, as the area decreases, storm duration becomes less important in determining rainfall gradients within the basin. Characteristics of the storm passing over the basin are perhaps of greatest importance here.

The relative weights of P and T in determining the characteristic of the area-depth curve are further illustrated in Table 7 where partial correlation coefficients between b and P and between b and T are given for the two areas.

TABLE 7

CORRELATION COEFFICIENTS*

Type	Items	Basin Areas 5.5 Sq. Mi.	100 Sq. Mi.
Partial	b vs. P	0.45	0.60
Partial	b vs. T	0.21	0.38
Multiple	b vs. P, T	0.55	0.65

While the results of the area-depth study are based on Illinois data, the relations should be generally applicable to shower-type rainfall in the Midwest or other areas having similar climate and topography.

Limited tests made on published area-depth data for the 8000-square mile Muskingum Basin [8] indicated that Equation (1) is not applicable to large basins. Chow [10] has suggested that an equation of the form

$$Y = a + b X^{1/2} + cX + dX^{3/2} + \ldots \qquad (7)$$

may be more applicable to larger basins, the number of higher terms depending on the basin size.

VARIATION OF POINT RAINFALL WITH DISTANCE

Introduction

The variation of rainfall with distance is an important factor in hydrologic and agricultural studies where analysis of precipitation effects is required. Due to the ordinary spacing of rain gages, estimates of point rainfall are frequently needed for locations which may be several miles from the nearest rain gage. The U.S. Weather Bureau [8] has presented some limited results for distances up to four miles, based upon data from five stations on Little Mill Creek in Ohio. Otherwise, little information on the subject is available.

As a partial solution to the problem of determining rainfall variability with distance, a study was undertaken, using storm data collected on two concentrated rain-gage networks. Data from 25 gages on the 100-square mile Panther Creek network (Fig. 3) for 1950-54 and from 50 gages on the 100 square mile Goose Creek network (Fig. 4) for 1952-54 were used in the study. These networks provided data for distances up to 11 miles from a total of 186 storms having point rainfall amounts ranging from 0.01 inch to 9.15 inches and areal mean rainfalls varying from a trace to 7.21 inches. Analysis was confined to warm season or shower-type rainfall occurring from spring to fall.

Analysis of Average Differences

The rain gage nearest the center of the area in each network was designated as the comparison gage. For each storm on each network, differences were calculated between the total storm rainfall at the comparison gage and that recorded at each of the other network gages. The data were first grouped according to distance from the comparison gage. At each distance, the differences between the various network gages and the comparison gage were further grouped in class intervals according to point rainfall amounts (storm size) at the comparison gage.

Graphical plots of the grouped data indicated one of the two following general equations would provide the best data fit:

$$\text{Log } E = k + l \text{ Log } P + m \text{ Log } D \qquad (1)$$

$$\text{Log } E = k + l\ P^{n} + m \text{ Log } D \qquad (2)$$

In Equations (1) and (2), E represents the average difference between the total storm rainfall at the comparison gage and at points located at distance D from the comparison gage, when the point rainfall recorded at the comparison gage is P. The regression constants are represented by k, l, m and n.

The data from both networks were then combined to obtain the empirical expressions:

$$\text{Log } E = -0.796 + 0.49 \text{ Log } P + 0.32 \text{ Log } D \qquad (3)$$

$$\text{Log } E = -1.359 + 0.51\ P^{0.5} + 0.31 \text{ Log } D \qquad (4)$$

where E and P are in inches and D is in miles.

Confidence Limits

For practical application, an estimate of storm rainfall at some point at a distance D from the comparison gage is more useful if an interval or range about the estimate can be determined with some measure of confidence that the estimate is within this range. The 95 per cent confidence bands are frequently used for this purpose. To establish confidence limits, an estimate of the sampling standard deviation is required. Consequently, expressions for an estimate of the standard deviations of the differences between the comparison gage rainfall and that occurring at various distances from the comparison gage were obtained from the same data used in deriving Equations (3) and (4). The expressions obtained are

$$\text{Log } s = -0.398 + 0.49 \text{ Log } P + 0.32 \text{ Log } D \qquad (5)$$

$$\text{Log } s = -0.961 + 0.51\ P^{0.5} + 0.31 \text{ Log } D \qquad (6)$$

The 95 per cent confidence bands for P were estimated by taking two times the standard deviation and adding and subtracting from the corresponding value of P.

Equation Comparisons

Multiple correlation coefficients of 0.74 and 0.78 were obtained from Equations (3) and (4), respectively. These correlation coefficients indicate there is little difference in the goodness of fit provided by these two equations. Graphical plots of the grouped data indicated that Equation (4) gives a somewhat better fit at high and low values of P, while Equation (3) fits best at the intermediate values of P.

A comparison is given in Table 8 of the E values obtained for various P values at a distance (D) of five miles. Only small differences in E are obtained from the two empirical equations below P values of three inches. Above three inches, Equation (4) produces appreciably higher values of E than Equation (3). However, only a small portion of the point rainfall observations from which the empirical equations were derived exceeded three inches.

TABLE 8

COMPARISONS BETWEEN EQUATIONS (3) AND (4)

	E (in.) at 5 Mi.	
P (in.)	Eq. (3)	Eq. (4)
0.10	0.08	0.10
0.25	0.14	0.13
0.50	0.19	0.17
1.00	0.27	0.23
2.00	0.38	0.38
3.00	0.45	0.55
5.00	0.59	1.00

From the hydrologic standpoint, at least, the variation of rainfall with distance in heavy storms is usually of greater interest than rainfall variations in light storms. Since Equations (3) and (4) predict significantly different values of E above P values of three inches, an effort was made to de-

termine which equation appears most applicable at high values of P. Although the data used in the study indicated Equation (4) provided a better fit at the high P values, it was desired to further test the relation with independent data.

For this purpose, data were used from two unusually heavy Illinois storms: the July 9, 1951 storm in North Central Illinois,[12] and the July 18-19, 1952 storm in Rockford and vicinity.[13] The Water Survey conducted extensive field surveys following these storms which provided adequate rainfall data for constructing detailed isohyetal maps. A rectangular grid overlay was used with the isohyetal maps from these two storms to obtain data on rainfall variations with distance. Each 3-mile interval on the grid overlay was considered an observation point and rainfall differences were obtained from the underlying isohyetal map at distances of three, six, and nine miles from each observation point in each of four directions - north, east, south and west. This method produced a total of 760 observations in the two storms. Point rainfall values at the observation points ranged from 2.0 to 10.6 inches. Equations (3) and (4) were then tested by determining how many of the 760 observations fell within the 95 per cent confidence bands for each equation. It was found that 94 per cent of these observations fell within the 95 per cent confidence bands of Equation (4) compared to 80 per cent for Equation (3). These results indicate that Equation (4) is more reliable for predicting differences in point rainfall with distance in heavy storms, as suggested earlier by graphical plots of the original data upon which the two empirical equations were based.

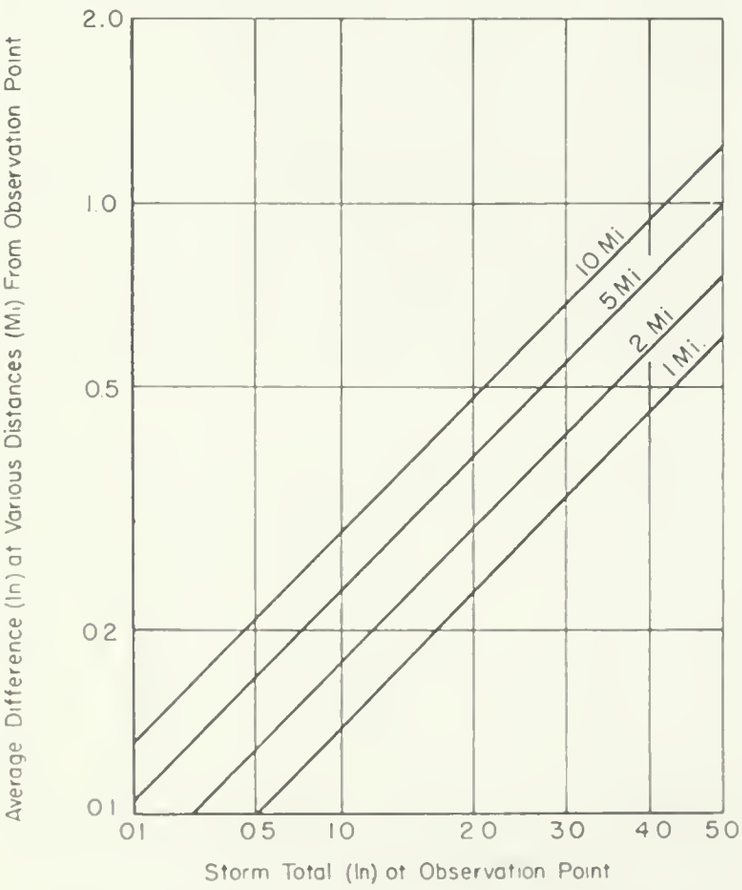


FIGURE 31 VARIATION OF POINT RAINFALL WITH DISTANCE

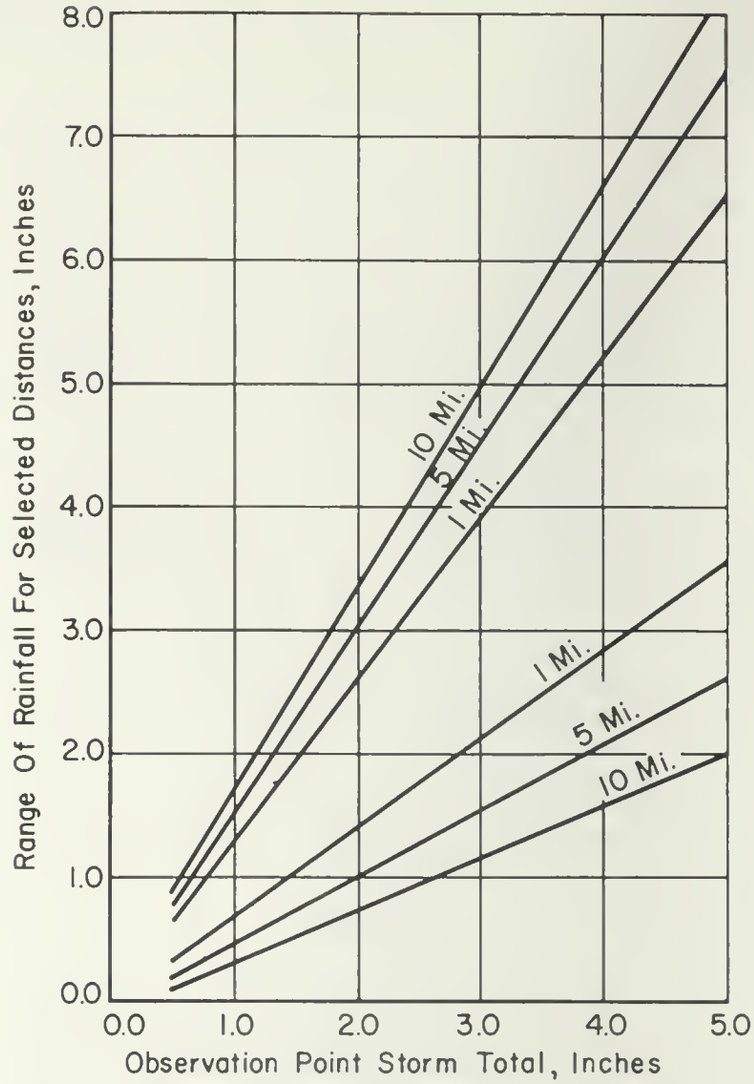


FIGURE 32 95 PER CENT CONFIDENCE RANGE FOR VARIATION OF POINT RAINFALL WITH DISTANCE

A graph derived from Equation (4) for predicting the average difference in storm rainfall between points at various distances is given in Figure 31. The 95 per cent confidence bands, based upon Equation (6), for selected values of D are presented in Figure 32. The great variability in shower-type rainfall is well illustrated in Figure 32. For example, within a distance of one mile from a gage recording a rainfall of one inch, differences up to 0.35 inch or 35 per cent are indicated by the 95 per cent limits; at five miles, differences up to 0.55 inch occur; and at 10 miles, differences reach 0.70 inch.

Consideration of Additional Variables

In an earlier, limited study using data for 33 storms over the Goose Creek network during 1953[14] analysis was performed to determine if a more efficient equation might be obtained by substituting other independent variables into Equation (1) in addition to P and D. The data had not been fitted to Equation (2) at the time of this analysis.

The only practical variables which can be used in an expression such as Equation (1) are those which can be read from the rain-gage chart at the observation gage. It was reasoned that consideration of storm duration (T), mean rate of rainfall (R_a), and maximum rate of rainfall (R_m) might

improve the prediction efficiency of expressions such as (3) and (5) which utilize only P and D. The maximum 5-minute rainfall rate for each storm was used to represent the maximum storm rate.

Multiple correlation coefficients between standard deviation (s) and various combinations of P, D, T, R_a, and R_m were computed to determine whether the correlation would improve from consideration of the three additional variables. The various coefficients are shown in Table 9. The squares of the coefficients, expressed in per cent, are also shown. These values indicate the relative efficiency of the various combinations of independent variables.

The results presented in Table 9 indicate that the three additional variables are about equal in efficiency. Since the product of R_a and T is equivalent to P, their use as variables in an expression already containing P appeared doubtful. However, to more thoroughly explore the problem, tests involving these two variables in conjunction with P were made. Table 9 shows that including R_a and T with P increased the efficiency by only eight per cent (42 to 50). The same efficiency was obtained by using R_a and T without P (last line of Table 9). Using T, R_a and R_m in conjunction with P and D increased the prediction efficiency by only 10 per cent.

Considering the relatively small improvement in efficiency shown by the more complex expressions and the additional work involved in getting data to use with them, Expressions (3) to (6) appear more practical for prediction purposes. Undoubtedly there are other factors which would be useful in increasing the efficiency of Equations (3) to (6). An important factor, for instance, is the storm path with reference to the location of the measurement gage. However, it is impossible to estimate the storm path from a single gage, and only factors which could be obtained from a single recording gage were investigated in this study.

TABLE 9

MULTIPLE CORRELATION COEFFICIENTS

Independent Variables	Multiple Correlation (C)	C^2 (%)
PD	0.65	42
PDR_m	0.71	50
PDT	0.70	49
PDR_a	0.70	49
$PDR_m T$	0.71	50
$PDR_m R_a$	0.71	50
$PDTR_a$	0.71	50
$PDR_m TR_a$	0.72	52
DTR_a	0.71	50

AREAL REPRESENTATIVENESS OF POINT RAINFALL

Introduction

The accuracy with which a point rainfall measurement represents the mean rainfall for areas of varying sizes in the vicinity of the point observation is pertinent to the design of rain-gage networks for various purposes and in the interpretation of data from existing networks. Information regarding the relation between point rainfall and areal mean storm rainfall is especially applicable to hydrologic problems.

The lack of concentrated rain-gage networks over areas of various size, which are necessary for deriving point-areal mean relations, has in the past limited development of empirical relations. Some limited data for areas of 375, 1500 and 8000 square miles have been provided by the U.S. Weather Bureau.[8]

As a partial solution to the above problem, rainfall data collected on several concentrated networks in central Illinois have been utilized to derive empirical relations between point and areal mean rainfall for areas ranging from 0.03 to 400 square miles. Point rainfall was measured at the center of each area and empirical equations developed for storm, weekly, and monthly periods. In addition, storm data for areas of 10 to 100 square miles were used to obtain an empirical relation between point and areal mean rainfall for conditions when the point rainfall is measured at a gage displaced from the areal center by various distances.

Available Data

Data used in this study were collected on the networks illustrated in Figures 2-8, comprising areas ranging from 0.03 to 400 square miles. In addition, the Goose Creek and Panther Creek networks were subdivided to provide data for areas of 10, 25, and 50 square miles. The various networks provided data for nine areal sizes from which a total of 1900 storm observations were available for derivation of empirical relations. The sampling scheme is illustrated in Figure 33, using the 1954 Goose Creek network.

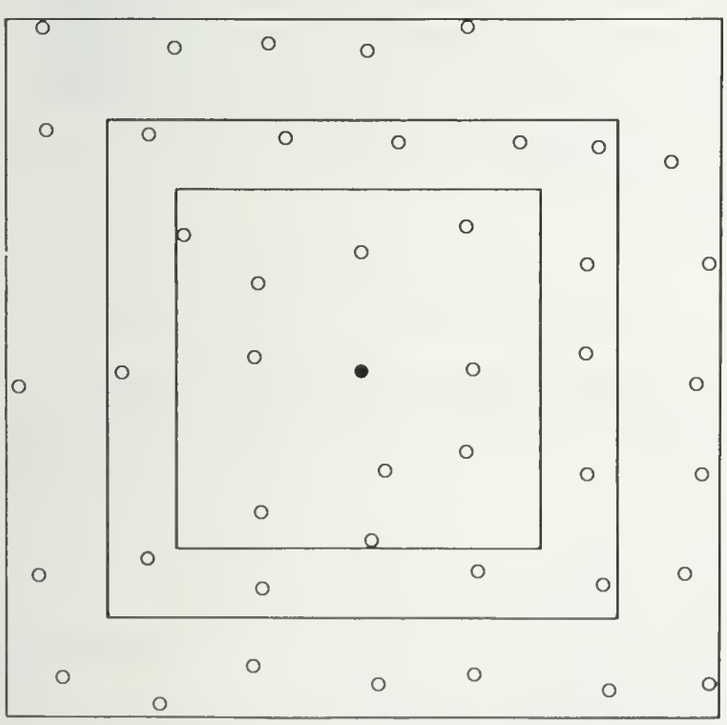


FIGURE 33 25, 50 AND 100 SQUARE MILE AREAS, GOOSE CREEK 1954

All data used in the study were collected during spring, summer, and early fall; consequently, the developed relations are most applicable to shower-type rainfall. While the results are based on Illinois data, they should be approximately representative for the Midwest, in general, and for other areas having similar climate and topography.

Accuracy of Areal Rainfall Estimates From a Centered Gage

Average Error. Various graphical plots were made of data for each areal size, relating point rainfall at the center of each area to the difference between the point observation and the areal mean rainfall as measured by all gages within that particular area. Inspection of these graphical plots indicated that one of the following general equations would fit the data best:

$$\text{Log } E = k + l \text{ Log } P \qquad (1)$$

$$\text{Log } E = k + l\ P^m \qquad (2)$$

In these expressions, P is the point rainfall for either the storm, weekly, or monthly observation at the center of a given area; E represents the difference between point and areal mean rainfall in the given area; and k, l and m are regression constants. Values for k and l for storm, weekly, and monthly rainfall are shown in Table 10 for each network based on Equation (1).

TABLE 10

REGRESSION CONSTANTS FOR STORM, WEEKLY AND MONTHLY EQUATIONS ON VARIOUS AREAS

Area	Storm		Weekly		Monthly	
(sq. mi.)	k	l	k	l	k	l
0.03	-2.155	0.46	-2.097	0.61	-2.301	0.72
0.60	-1.538	0.46	-1.469	0.61	-1.347	0.38
4	-1.377	0.47	-1.260	0.73	-1.252	0.86
10	-1.377	0.36	-1.229	0.62	-1.268	0.84
25	-1.222	0.52	-1.051	0.68	-1.252	0.78
50	-1.167	0.51	-1.060	0.78	-1.174	0.80
100	-1.131	0.50	-1.065	0.66	-1.131	0.74
280	-0.914	0.45	-0.783	0.69	---	---
400	-0.932	0.53	---	---	---	---

The values of l vary at random, whereas the k values show an increasing trend with area. Consequently, it was felt that the data from all networks could be combined into a single expression of the form:

$$\text{Log } E = k_1 + l_1 \text{ Log } P + n \text{ Log } A \qquad (3)$$

relating the measurement error of areal mean rainfall to the point rainfall (P) recorded at the

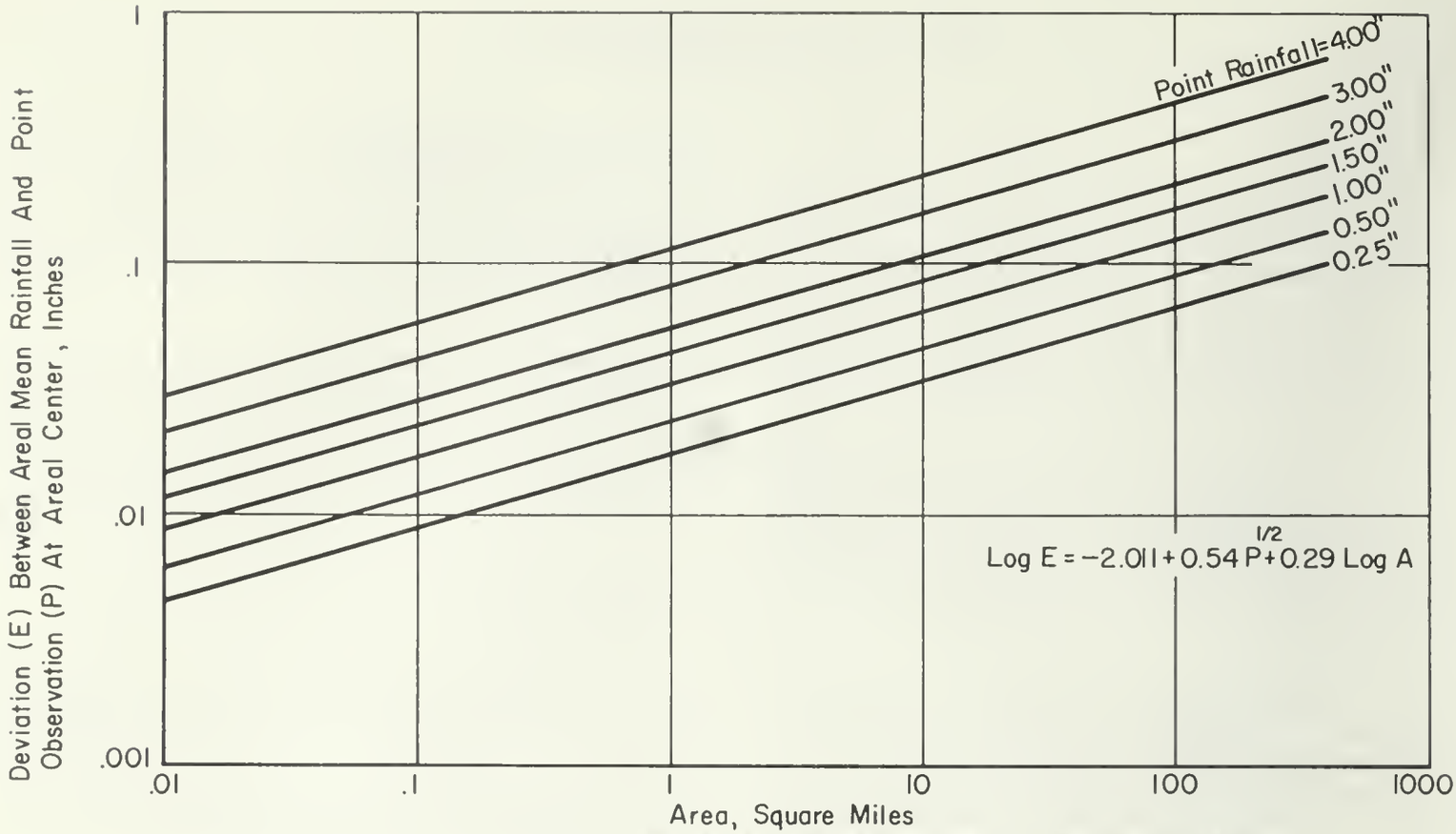


FIGURE 34 STORM RELATION BETWEEN POINT AND AREAL MEAN RAINFALL BASED ON AVERAGE DEVIATIONS

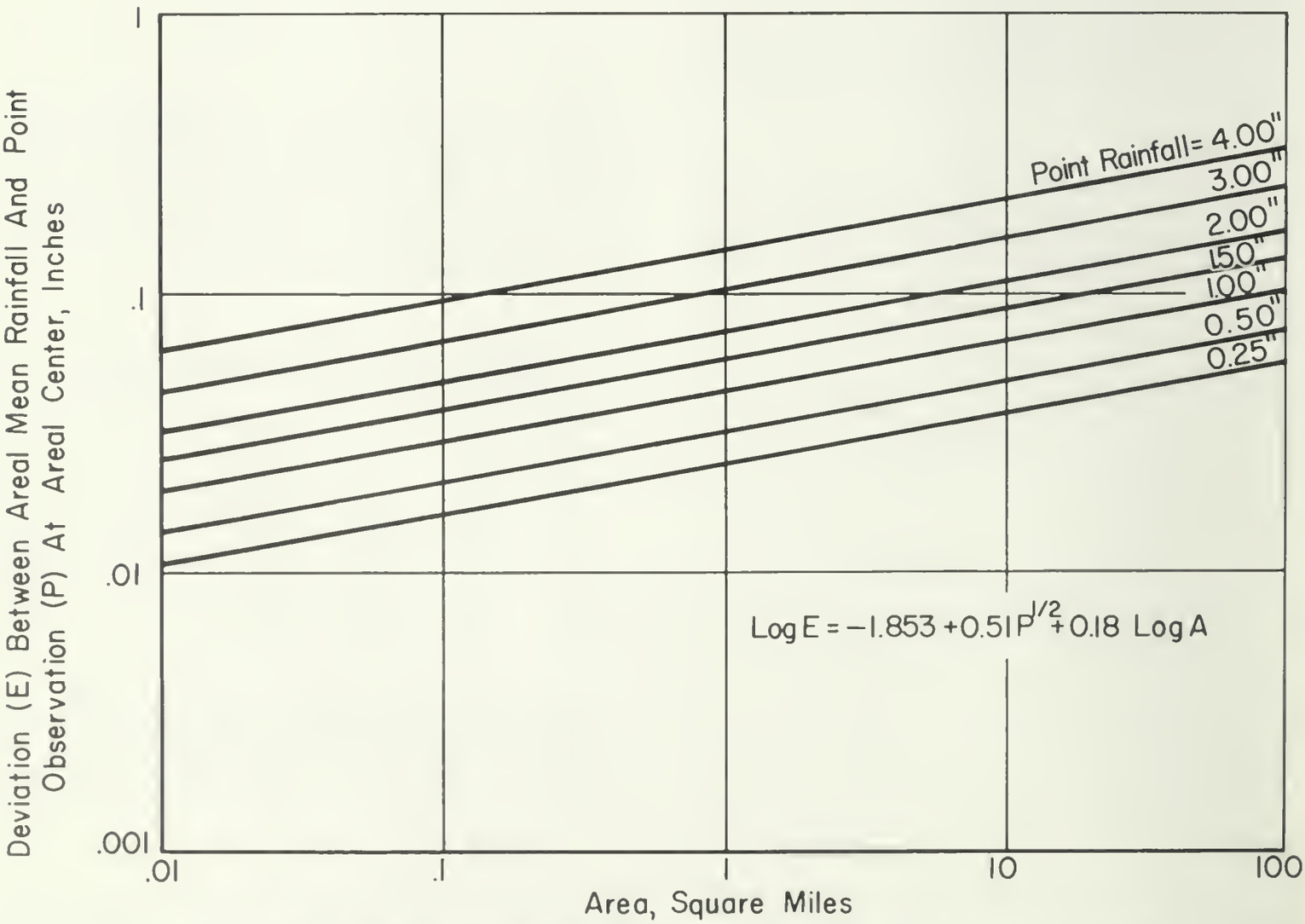


FIGURE 35 WEEKLY RELATION BETWEEN POINT AND AREAL MEAN RAINFALL BASED ON AVERAGE DEVIATIONS

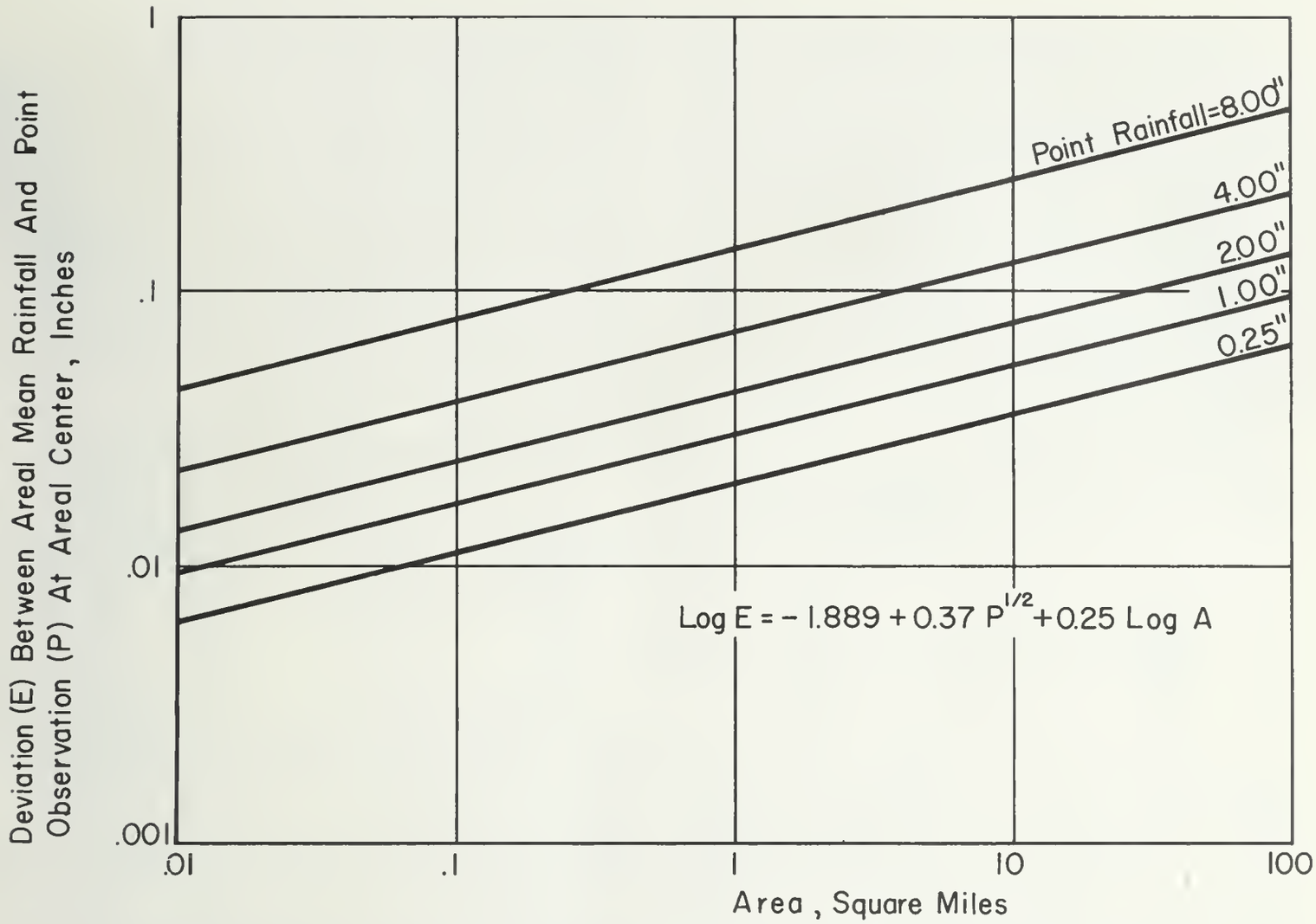


FIGURE 36 MONTHLY RELATION BETWEEN POINT AND AREAL MEAN RAINFALL BASED ON INDIVIDUAL DEVIATIONS

center of the area and to the size of the area (A). The constants k_1, l_1 and n were determined by the method of least squares, and the following empirical equations for storm, weekly, and monthly rainfall were obtained.

$$\text{Log } E = -1.341 + 0.50 \text{ Log } P + 0.29 \text{ Log } A \quad \text{(Storm)} \quad (4)$$

$$\text{Log } E = -1.285 + 0.58 \text{ Log } P + 0.20 \text{ Log } A \quad \text{(Weekly)} \quad (5)$$

$$\text{Log } E = -1.559 + 0.71 \text{ Log } P + 0.26 \text{ Log } A \quad \text{(Monthly)} \quad (6)$$

For comparison purposes, the data were next fitted to an expression of the form:

$$\text{Log } E = k_2 + l_2 P^m + n_1 \text{ Log } A \quad (7)$$

in which E, P, and A represent the same quantities as in Equation (3), and k_2, l_2, m, and n_1 are again regression constants. The resulting empirical equations are:

$$\text{Log } E = -2.011 + 0.54 P^{0.5} + 0.29 \text{ Log } A \quad \text{(Storm)} \quad (8)$$

$$\text{Log } E = -1.853 + 0.51 P^{0.5} + 0.18 \text{ Log } A \quad \text{(Weekly)} \quad (9)$$

$$\text{Log } E = -1.889 + 0.37 P^{0.5} + 0.25 \text{ Log } A \quad \text{(Monthly)} \quad (10)$$

Multiple correlation coefficients were then determined for Equations (4), (5), (6), (8), (9), and (10) and are presented in Table 11. These correlation coefficients indicate there is no significant difference in the goodness of fit provided by Equations (3) and (7). The smaller correlation coefficient obtained with the monthly data is due primarily to the fact that analysis was performed on ungrouped data, whereas the storm and weekly analyses were performed on grouped data. The number of individual monthly observations did not permit grouping. There were only 255 monthly values compared to 663 weekly totals and 1900 storm observations.

Graphical plots of the grouped data used in Equations (4), (5), (8), and (9) indicated that Equations (8) and (9) provide a somewhat better fit at high and low values of P. A similar occurrence was noted between these two general types of equations in the study of the variation of point rainfall with distance discussed in the previous section of this bulletin.

TABLE 11

MULTIPLE CORRELATION COEFFICIENTS

Equation Number	Correlation Coefficient	Rainfall Type
(4)	0.89	Storm
(8)	0.90	Storm
(5)	0.85	Weekly
(9)	0.85	Weekly
(6)	0.62	Monthly
(10)	0.60	Monthly

A comparison of the E values obtained for various P values from Equations (4) and (8) for storm rainfall on an area of 100 square miles is presented in Table 12. Only small differences in E are obtained from the two empirical equations below P values of three inches. Above three inches, Equation (8) produces appreciably higher values of E than Equation (4). As mentioned previously, indications are that empirical equations derived from Expression (7) fit heavy rainfall data better. Since heavy rainfall is of prime interest to the hydrologist, further analytical efforts have been concentrated on this form of equation. Equations (8), (9) and (10) are represented graphically in Figures 34, 35, and 36.

TABLE 12

COMPARISONS BETWEEN EQUATIONS (4) AND (8)

P (in.)	E (in.) for 100 Square Mile Area Eq. (4)	Eq. (8)
0.10	0.04	0.06
0.25	0.07	0.07
0.50	0.10	0.09
1.00	0.14	0.13
2.00	0.20	0.22
3.00	0.25	0.32
5.00	0.32	0.60

A comparison of E values obtained from the storm, weekly and monthly equations is shown in Table 13. As expected, all three equations show that the average difference between point and areal mean rainfall (E) increases as the storm size, represented by the point measurement at the areal center, increases. For a point rainfall measurement of a given magnitude, a trend exists for E to decrease with increasing sampling time as represented by storm, weekly and monthly values. This trend becomes more pronounced as the point rainfall at the areal center increases. It would appear, therefore, that a centered gage measures areal mean rainfall with an increasing degree of accuracy as the sampling period increases. Such a trend is to be expected. The weekly rainfall is usually the result of several storms and the monthly total ordinarily contains more storms than the weekly total. In the absence of significant topographic or climatic features, the areal distribution of rainfall will tend to become more uniform as the number of storms increases, because individual storms tend to travel in various paths across a given area.

TABLE 13

COMPARISON BETWEEN STORM, WEEKLY AND MONTHLY RELATIONS

Point Rainfall (in.)	Av. Difference (in.) Between Point and Mean Rainfall for 100-Square Mile Area Storm	Weekly	Monthly
0.25	0.07	0.06	0.06
1.00	0.13	0.10	0.10
2.00	0.22	0.17	0.14
4.00	0.45	0.34	0.22

Confidence Limits. An estimate, P, of the areal mean rainfall becomes more meaningful when some measure is made of the possible error in the estimate. It is desirable to determine an interval about P with some measure of confidence that the areal mean is in that interval. To establish confidence limits, an estimate of the sampling standard error was required. This was made from 14 groups of storms having about the same P values. These classes, expressed in inches, are shown in Table 14 A standard deviation was computed for each of the class intervals in Table 14. The constants k, l, m and n were again deter-

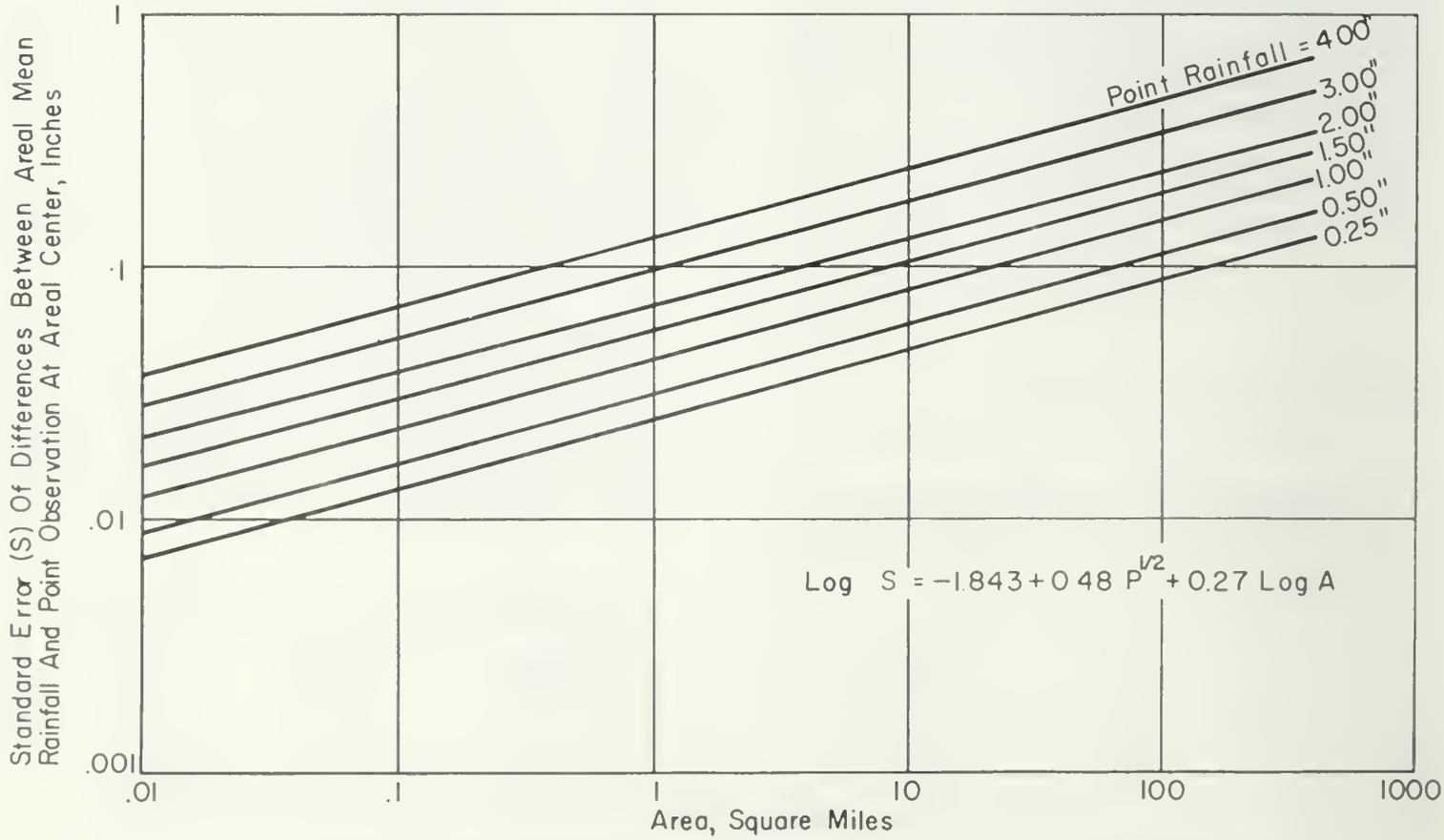


FIGURE 37 STORM RELATION BETWEEN POINT AND AREAL MEAN RAINFALL BASED ON STANDARD ERRORS

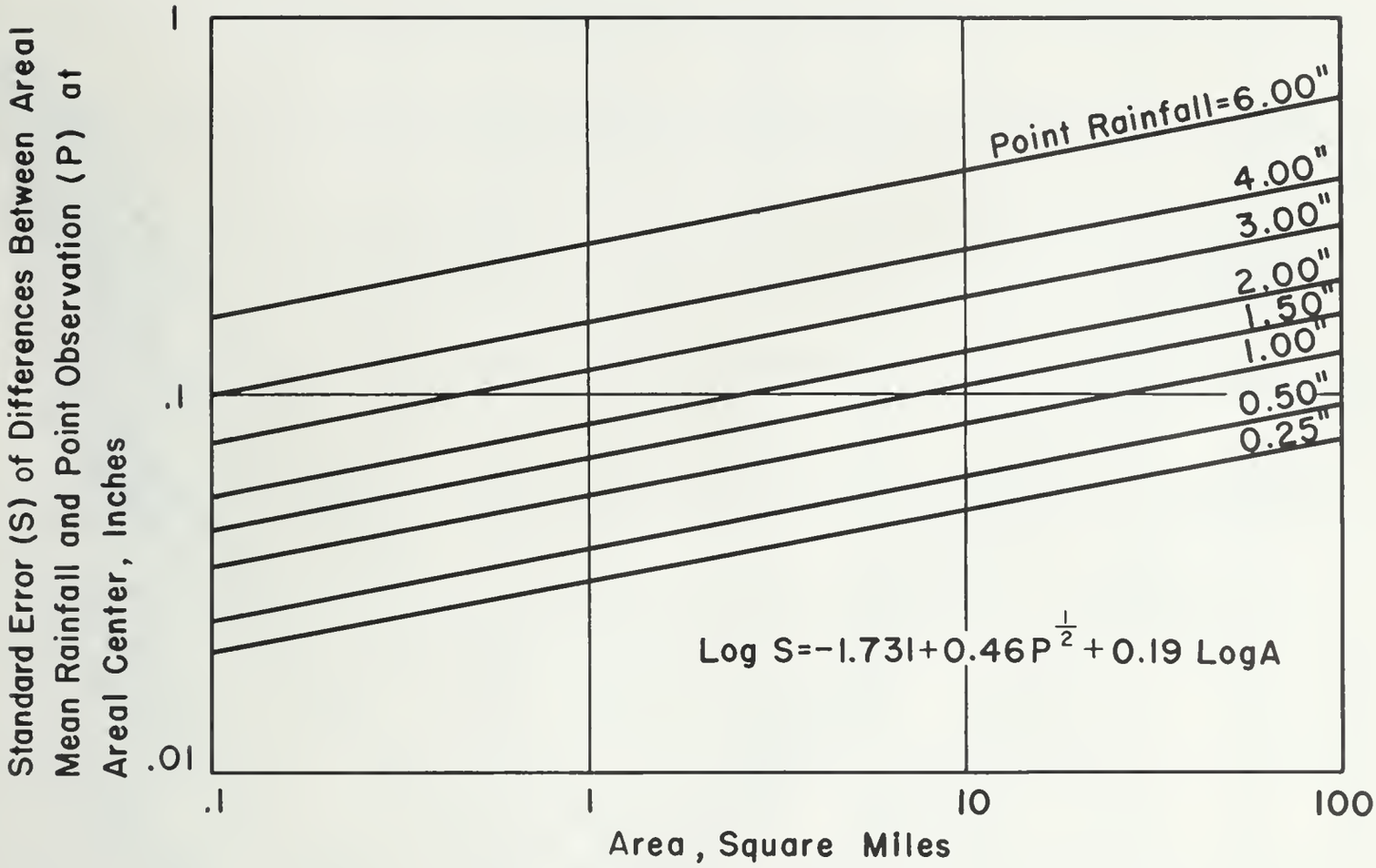


FIGURE 38 WEEKLY RELATION BETWEEN POINT AND AREAL MEAN RAINFALL BASED ON STANDARD ERRORS

mined for equations of the form represented by Expression (7). The resulting equations for storm and weekly rainfall are:

$$\text{Log } s = -1.843 + 0.48\ P^{0.5} + 0.27 \text{ Log } A \quad \text{(Storm)} \quad (11)$$

$$\text{Log } s = -1.731 + 0.46\ P^{0.5} + 0.19 \text{ Log } A \quad \text{(Weekly)} \quad (12)$$

In these equations, s (inches) represents the best estimate of the standard error, P (inches) represents the mid-point of the class intervals, and A represents the area in square miles. Multiple correlation coefficients for Equations (11) and (12) were 0.85 and 0.82, respectively. Figures 37 and 38 show a graphical relation between A and s for several P values for storm and weekly rainfall. An insufficient number of monthly totals prevented the determination of a monthly relationship.

The 95 per cent confidence limits for storm and weekly areal mean rainfall were estimated by computing p ± 2 s from Equations (11) and (12). The resulting limits are presented for several values of A in Figures 39 and 40. Other confidence limits may be determined if desired. For example 70, 80, 90 and 99 per cent confidence limits for storm and weekly areal mean rainfall may be estimated by multiplying s from Equations (11) and (12) by 1.05, 1.30, 1.67 and 2.66, respectively, and adding and subtracting from P.

TABLE 14

STORM GROUPS (IN.) USED IN STANDARD ERROR COMPUTATIONS

0.01-0.10	0.41-0.50	0.81-0.90	2.01-3.00
0.11-0.20	0.51-0.60	0.91-1.00	3.01-4.00
0.21-0.30	0.61-0.70	1.01-1.50	
0.31-0.40	0.71-0.80	1.51-2.00	

Areal Rainfall Estimates from an Off-Center Gage

Frequently, rain gages are not located at or near the center of the area of interest. The nearest gage may even be outside the boundary of the area. It is logical to expect that a rain gage which is located at a distance from the areal center will on the average give a less accurate estimate of the areal mean than a gage located at the center. The following paragraphs summarize an analysis to determine the accuracy of storm areal mean rainfall estimates from gages located at various distances from the center of several areas of different sizes.

Data from areas of 10, 25, 50, and 100 square miles were analyzed. Analysis techniques were similar to those used in comparing areal mean rainfall with the rainfall amount obtained from a centrally-located gage. An examination of graphical plots of the data indicated that an expression of the form

$$\text{Log } E = a + b\ P^{c} + d \text{ Log } A + e \text{ Log } D \quad (13)$$

followed the trend of the data satisfactorily. In this expression, E represents the average of the differences, A is the network area, D is distance from the areal center, P represents rainfall amounts at

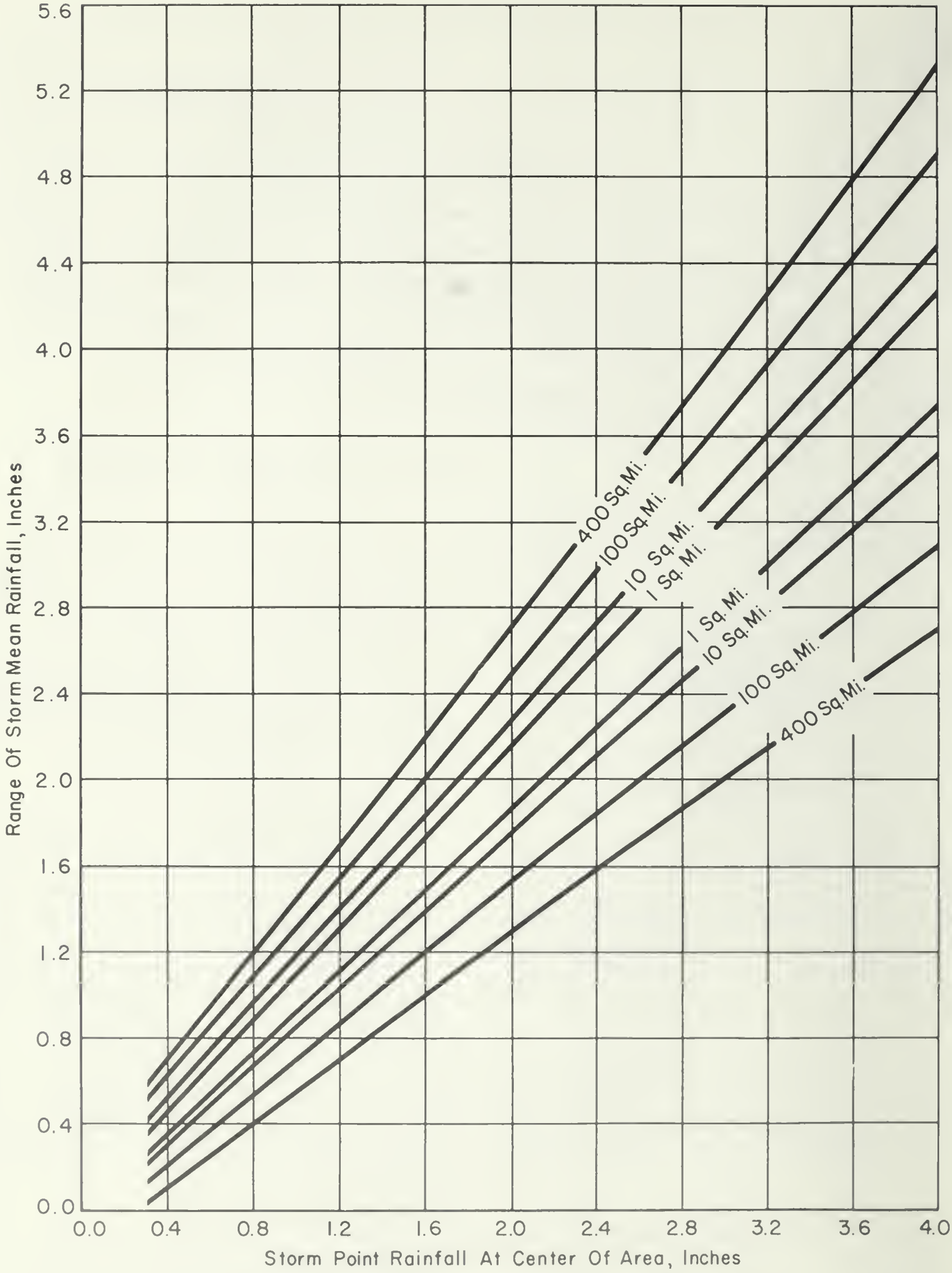


FIGURE 39 95 PER CENT CONFIDENCE RANGE FOR STORM RAINFALL

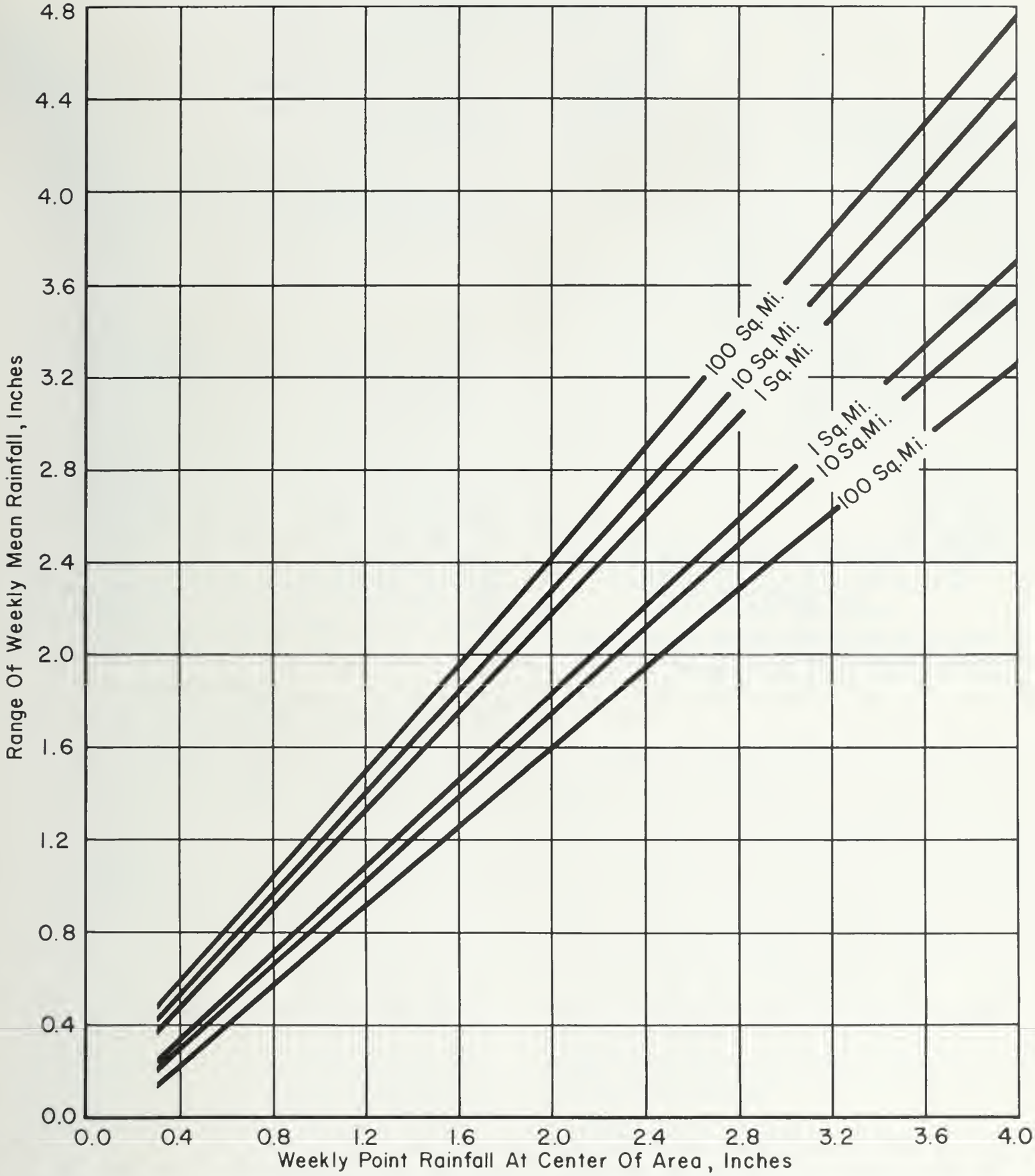


FIGURE 40 95 PER CENT CONFIDENCE RANGE FOR WEEKLY RAINFALL

the observation gage; a, b, c, d, and e are regression constants.

The equation obtained by fitting Expression (13) to the data is:

$$\text{Log } E = -1.411 + 0.51\ P^{0.5} - 0.01 \text{ Log } A + 0.37 \text{ Log } D \qquad (14)$$

In this equation E and P are in inches, A is in square miles and D is in miles. The relation is illustrated for an area of 100 square miles in Figure 41.

Within the limits of the data used, the magnitude of the coefficient of Log A suggests that area was not an important variable in the analysis. This

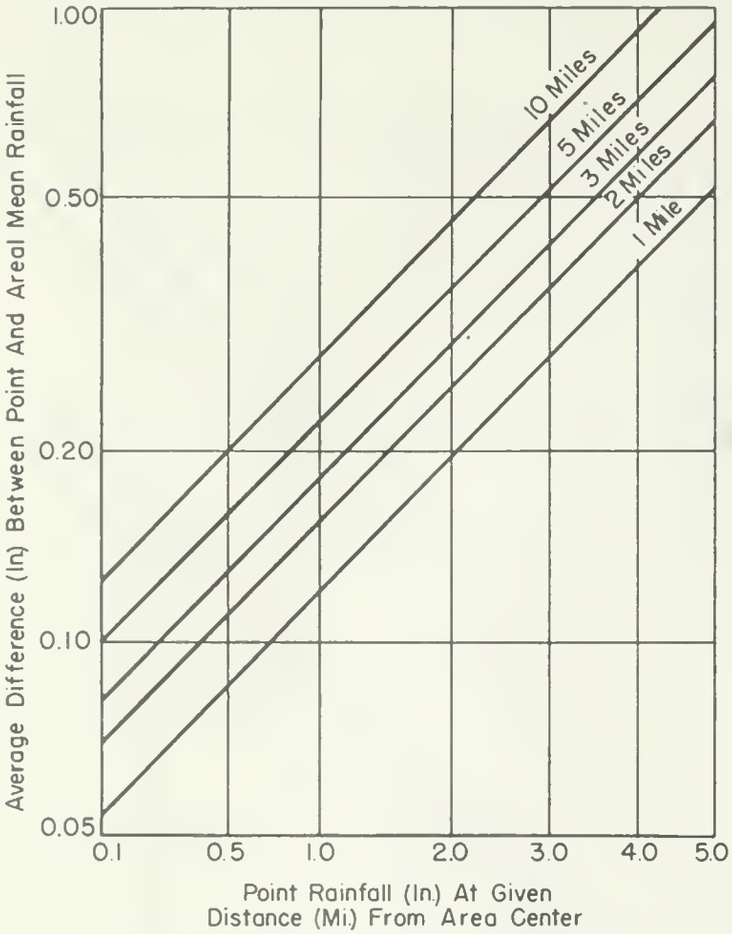


FIGURE 41 EFFECT OF GAGE LOCATION ON SINGLE GAGE ESTIMATES OF MEAN RAINFALL ON 100 SQUARE MILE AREA

result is rather difficult to rationalize. However, it may be due to the fact that a gage located at any given distance from the center of two areas of different size is relatively closer to the center of the larger area. Consequently, the gage may be in an equally good position for sampling the mean rainfall in both areas.

An empirical equation for the standard deviation was determined for use in estimating confidence limits. This equation is:

$$\text{Log } s = -1.179 + 0.41\ P^{0.5} - 0.01 \text{ Log } A + 0.30 \text{ Log } D \qquad (15)$$

where s is in inches and P, A, and D have the same definition and units of measurement as they had in Equation (14). The 95 per cent confidence limits for various values of P, A, and D may be computed from $P \pm 2\ s$.

Since A contributes very little to the value of E and s in Equations (14) and (15), the area variable could be omitted without significant error within the limits of area tested in this study. These equations would then be of the same form as Equations (4) and (6) in the variation of rainfall with distance study discussed in the previous section of this bulletin. The coefficient of $P^{0.5}$ is the same and the coefficient of Log D nearly equal in both studies. These results suggest that the estimators P and D give practically the same measurement error in both analyses.

RELIABILITY OF STORM MEAN RAINFALL ESTIMATES

Introduction

In the previous section, the accuracy of areal rainfall estimates obtained from a single gage at or near the center of an area was investigated. However, an estimate of average rainfall may be required for an area on which more than one gage occurs. Consequently, an analysis was made to determine an estimate of both the average error and the standard error involved in measuring areal mean rainfall by means of various gage densities within several areas of different size.

Data Used

Data from the Panther Creek, Goose Creek, El Paso and east central Illinois networks were included in the analysis. Only shower and thunderstorm rainfall were considered. The 100-square mile Goose Creek network was subdivided into networks comprising 25 and 50 square miles for the 1952, 1953, and 1954 seasons. Two 200-square mile networks were chosen; one from the 280-square mile El Paso network (Fig. 2) operated in 1948 and 1949, and the other from the 400-square mile east central Illinois network (Fig. 5) operated in 1955 and 1956. These data permitted an analysis for areas of 25, 50, 100, 200, and 400 square miles.

Sampling Procedures

More than one sampling plan was considered to ascertain the most applicable plan for sampling areal mean rainfall. Three sampling plans were tried on a 100-square mile network. These included the random start, a plan which combined centrally located samples with random start samples, and a single best-centered sampling plan.

A random sampling procedure (not used in this study) allows the selection of gages in each sample to be determined entirely by chance. A stratified random sampling plan provides a more consistently uniform distribution of gages in each sample than that obtained by a purely random plan, but allows the selection of gages to be determined more by chance than does a random start sampling plan, for example. Sampling plans which involve the selection of centrally located gages in contiguous areas require the omission of a considerable number of observations from the analysis. A random start systematic sampling procedure provides a plan for spreading the sample observations over the network, and at the same time, permits the use of data from all gages.

Random Start. The sampling procedure for the random start systematic sampling plan can be illustrated by reference to Figure 42. When the total number of gages in a network is 48 and it is desired to take samples of 3 gages, the 48 gages are divided into 16 contiguous groups of 3 gages each. According to the random start plan, a starting position is selected in one group of gages. One gage in approximately the same location is automatically designated in each of the other groups to complete the observations in each sample. In a network of 48 gages, there are three possible samples of 16 which are treated as having equal probability of being an actual sample of 16 gages. Other sample sizes may be designated in a similar manner.

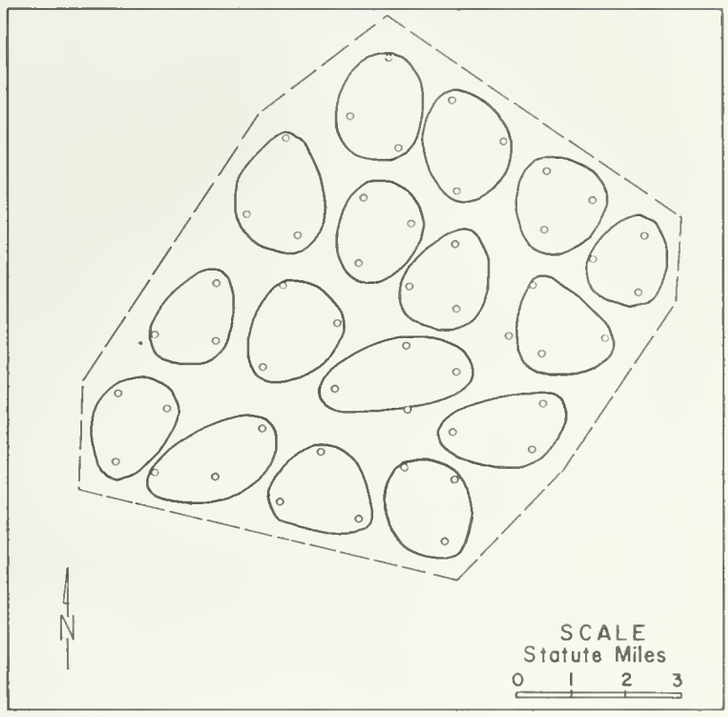


FIGURE 42 GROUPS OF GAGES FROM WHICH 3 RANDOM START SYSTEMATIC SAMPLES OF SIZE 16 WERE SELECTED

Combined Sampling Plan. The random start sampling plan does not provide for a single gage sample. When the sample size is decreased to one gage, the random start plan of dividing the network into groups of contiguous gages becomes the same as selecting one observation at random. The variance of these estimates would be equivalent to the purely random sampling variance for single gage samples. There is also a tendency for the random start systematic sampling plan to approach random sampling for other small samples, because the spread of observations in each sample becomes less uniform over the areas as the sample size decreases. This feature of the random start plan allows sampling errors for the small sample sizes which are somewhat larger than would be observed normally in actual practice, for the reason that gages in an operating network usually approach a uniform distribution.

When one is designing a network, each rain gage would logically be placed relatively close to the center of the area to be gaged. A centrally-located gaging plan should permit sampling errors of a magnitude less than a plan in which gage locations were left partially or wholly to chance. This argument may be advanced for random start systematic sample sizes of 2, 3, 4, and possibly 6 and 8 gages in 100 square miles. Consequently, a sampling plan was tried which involved sample sizes of 1, 2, 3, 4, 6, and 8 which were obtained from a centrally located plan as illustrated in Figures 43-A through 43-F.

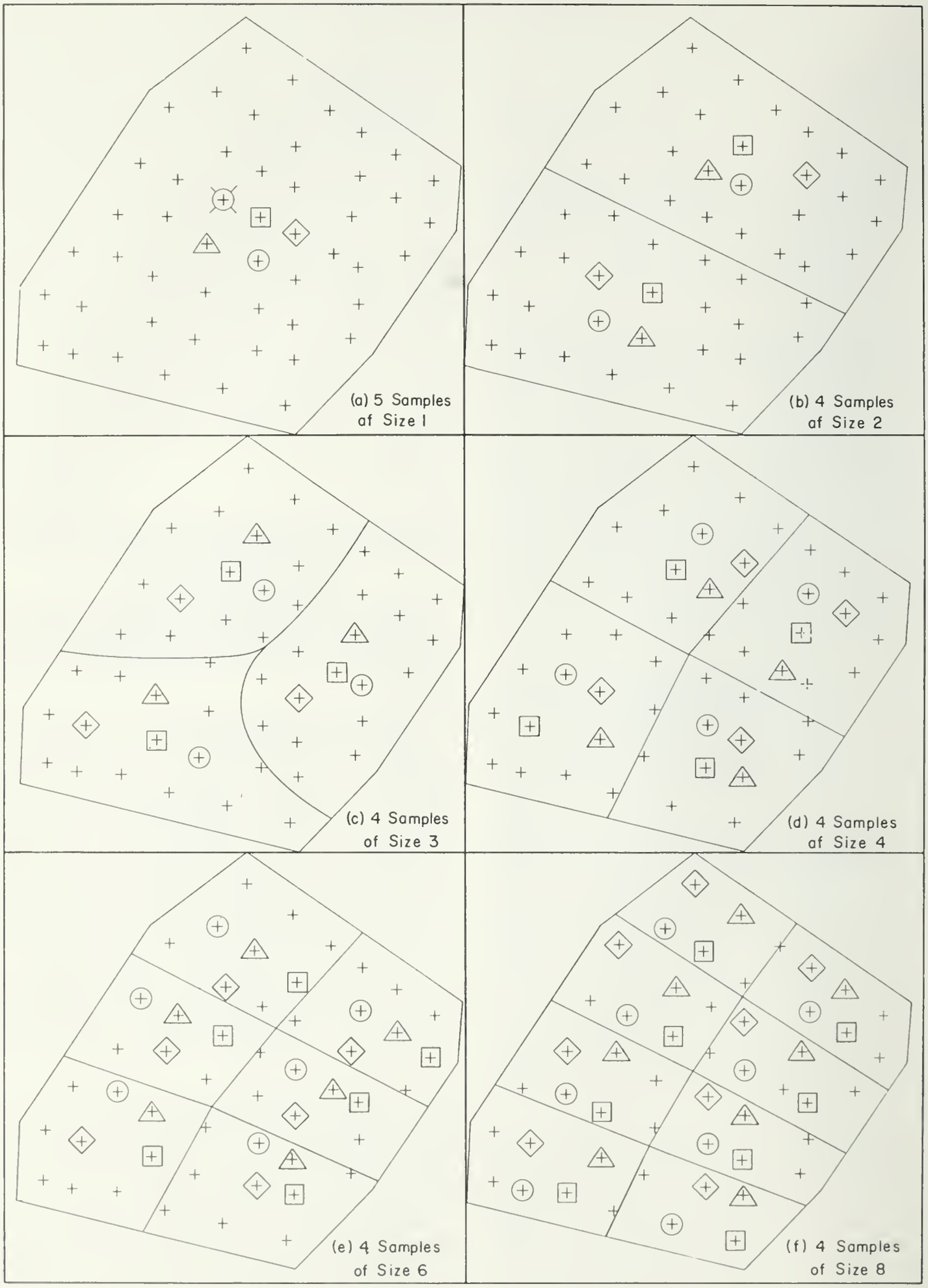


FIGURE 43 COMBINED SAMPLING PLAN FOR SAMPLES OF SIZE 1, 2, 3, 4, 6 AND 8

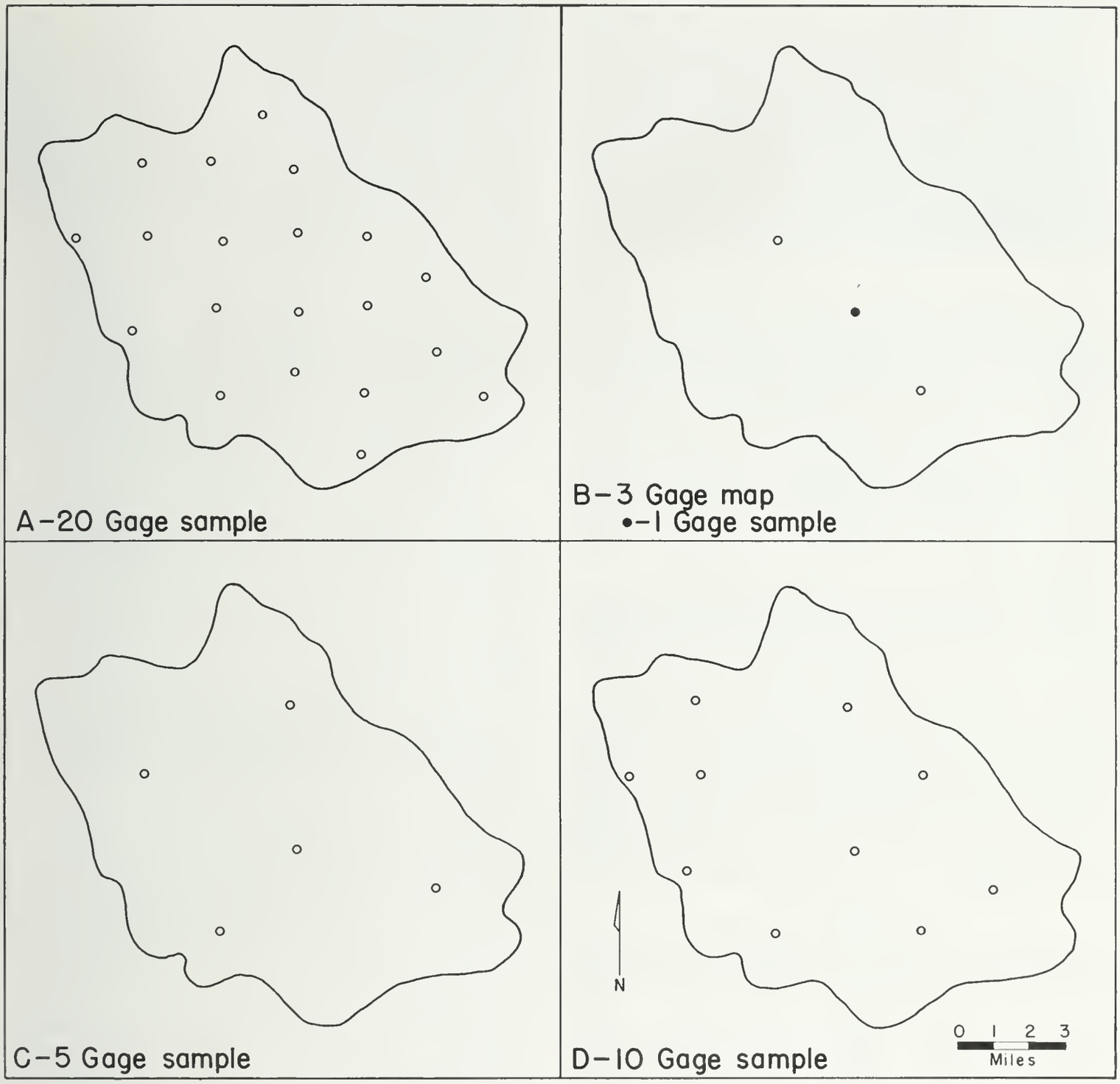


FIGURE 44 BEST-CENTERED SAMPLING PLAN FOR PANTHER CREEK NETWORK

For samples of size 1, five gages were selected within 1-1/4 miles of the geographic center of the network to represent the possible samples for this size. For samples of size 2, 3, 4, 6, and 8 the network was divided into 2, 3, 4, 6, and 8 sections, respectively. Gages located near the centers of these sections were designated as observation stations. As the sample size increased beyond eight, it became impossible to select samples which would be more centrally located than the random start samples. Consequently, the random start systematic plan was used for designating samples having more than eight gages.

Best-Centered. The third sampling plan which was selected was that defined as the one best sub-sample for each gage density, that is, approximately a uniform grid or centered systematic sample. This sampling plan has one advantage over either of the other two described, because it more closely approaches the practical network design. The best-centered plan has the advantage of providing data that allow for easy calculation of either the average or the standard error of mean rainfall. However it has the disadvantage of being dependent on a single subjective selection of a best sample. This plan does not utilize information that was gathered at all the other gages of the network except in the estimation of the network average. Gage locations used for this sampling plan are shown in Figures 44A through 44D.

Analytical Procedure

The main problem in the anlysis was the estimation of the true variance of the estimated average precipitation, P, about the true average precipitation for the network area under consideration. An estimate of this variance leads to an expected measure of the error involved in sampling with each different gage density. The computation consisted mainly of two steps. The first step was the determination of the deviations from the network mean by the following formula

$$d = (P - \bar{P}) \qquad (1)$$

where d represents the absolute value of the difference between P, the sample mean rainfall from a sample of N gages, and $\bar{P}$, the areal mean rainfall based on the total number of gages in the network.

The second step in determining an estimate of the error involved the fitting of a regression system to the deviations. The regression lines provide values which are designated as the estimate of the error of P about $\bar{P}$. A discussion of the regression system follows.

From graphical plots of the data it was observed, in general, that the deviation of the sample estimate from $\bar{P}$ increased as P increased, although there was considerable fluctuation in this upward trend. Also, the deviations generally increased as the number of observations (rain gages) in the sample decreased. Undoubtedly, there are many other factors which contribute to the variability of these deviations from storm to storm, such as the (1) meteorological factors causing the storm, (2) location of the storm core with respect to the center of the network, (3) duration of the storm, and (4) rate of rainfall. However, it is difficult to express (1) and (2) quantitatively and P is a function of (3) and (4).

Graphical plots indicated that the average magnitude of the deviations was also a function of N. Mathematical expressions of the form

$$\text{Log } s = a + b \text{ Log } P + c \text{ Log } N \qquad (2)$$

and

$$\text{Log } s = a + b\, P^m + c \text{ Log } N \qquad (3)$$

appeared to provide the best fit to the data. In these expressions, s represents the standard deviation of the absolute values of d from Expression (1). The goodness of fit of Expression (2) was determined in preliminary analyses.

Results of Analysis on 100-Square Mile Network

Random Start. When Expression (2) was fitted to the deviations obtained by the random start systematic sampling plan, the multiple correlation coefficient was 0.52. Although this correlation is significant, a correlation of greater degree is desirable for predicting the expected deviation between the sample and the areal mean rainfall.

It was felt that a part of the variation not accounted for by the regression system was due to variation in the distribution of rainfall over the network among storms of similar mean rainfall. A practical quantitative measure of areal rainfall distribution which can be included in Expression (2) is difficult to obtain. It was thought that the duration of rainfall might partially reflect the areal distribution, since the distribution of point rainfall amounts may depend upon the storm duration factor. Consequently, the duration factor, T, was added to Expression (2) and the goodness of fit of an expression of the form

$$\text{Log } s = a_1 + b_1 \text{ Log } P + c_1 \text{ Log } N + e_1 \text{ Log } T \qquad (4)$$

was determined. Fitting Expression (4) to the data resulted in a multiple correlation of 0.53. It is evident that the duration variable did not significantly increase the efficiency in predicting the expected deviation. Consequently, it was felt that adding the duration factor to the regression system did not warrant the extra work involved.

Combined Sampling Plan. When Expression (2) was fitted to the data for the deviations obtained from the combined sampling plan, the resulting multiple correlation coefficient was 0.70. This increase in the correlation coefficient from 0.52 for the random start plan apparently reflects a general increase in stability of the sample estimates from the more uniform distribution of gages for the small sample sizes.

Another independent variable was then introduced into the previous regression system in order to examine more thoroughly the estimate of the sampling variation. An expression of the following form

$$\text{Log } s = a_2 + b_2 \text{ Log } P = c_2 \text{ Log } N + e_2 \text{ Log } s_{\bar{P}} \qquad (5)$$

was fitted to the data, where $s_{\bar{P}}$ is the standard deviation of the entire network of gage readings, e is a constant, and the other symbols represent the same quantities as they did in previous discussions. The $s_{\bar{P}}$ variable represents the best available estimate of variation in the areal precipitation pattern. The sampling error for various sample sizes should depend considerably upon the variability of the precipitation pattern. When Expression (5) was fitted to the data the multiple correlation was increased from 0.70 to 0.84. The simple correlation coefficients between log s and log P, log N, and log $s_{\bar{P}}$ were 0.40, 0.58 and 0.65, respectively.

From the preceding correlation coefficients it is evident that the expected sampling error is highly dependent upon the variation in rainfall, as measured by the standard deviation of a relatively dense network of rain gages. It is also evident, as would be expected, that the sampling error has a higher degree of correlation with variation of rainfall than with either mean rainfall or gage density.

It should be noted that the insertion of the standard deviation of point rainfall totals from all gages into the equation produces an estimate that is not practical from the hydrologic standpoint. Dense networks of rain gages are seldom used except in research. The hydrologist frequently needs a means of estimating the sampling error for mean rainfall

amounts which have been obtained from a sparse network of gages. Consequently, the standard deviation, s_P of point rainfall amounts included in each sample was inserted into Expression (5) in place of $s_{\bar{P}}$. For example, if five gages were included in a sample, s_P would be the standard deviation of the five point rainfall amounts. The magnitude of s_P is a measure of the precision of a sample mean rainfall value. This variable should have an influence upon estimates of the sampling error for areal mean rainfall values. Inserting this variable into the expression resulted in a multiple correlation of 0.71. This is a very small increase over the 0.70 coefficient which was obtained from Expression (5). Consequently, it must be concluded that the s_P variable is not useful as a third independent variable. The inefficacy of s_P probably reflects the fact that the variation of sample standard deviations within storms approximated the variation in the standard deviations of a particular set of gage readings from storm to storm. Effectiveness of the sample standard deviation variable increased as sample size increased. However, the larger variation associated with small sample sizes offset most of the effect of the standard deviations for larger sample sizes. The efficiency of standard deviation as a third independent variable reached a maximum when all gages were included, as was the case when the $s_{\bar{P}}$ variable was used.

Best-Centered. A standard deviation of d values from Expression (1) was obtained by an adjustment similar to that used by Linsley and Kohler in their analysis of data from a concentrated network near Wilmington, Ohio.[15] Storm data were grouped into classes on the basis of sample mean rainfall as measured by a sample of N gages. A standard deviation was computed for each class. Expression (2) was then fitted to the data, using the standard deviation for each class as the dependent variable. The sample size and the mid-point of the rainfall for each class were the independent variables. A multiple correlation coefficient of 0.86 was obtained with this regression system.

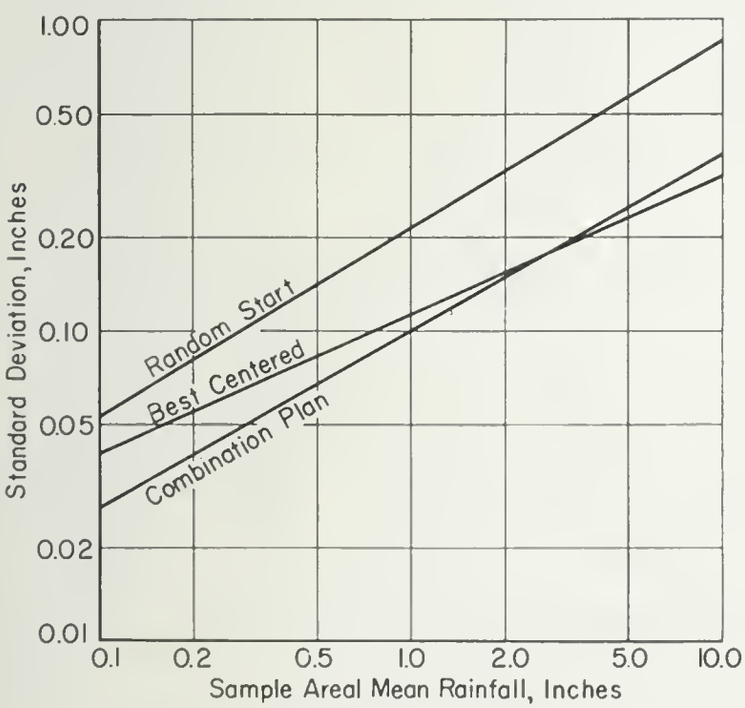


FIGURE 45 COMPARISON OF STORM SAMPLING PLANS ON A 100 SQUARE MILE AREA

Comparison of Results from Three Sampling Plans

A comparison of sampling error equations for the three sampling plans can be made by reference to Figure 45. It is evident that the random start plan will predict considerably higher sampling errors than either of the other plans. The other two sampling plans resulted in errors which are very similar in magnitude within the range of data used. The a and c values for the best-centered and the combination plan are almost identical. The differences between the b values could be attributed to sampling variation.

Results of Analysis for Areas of Different Size

Average Error. The best-centered sampling plan approaches the practical situation which hydrologists must contend with in using the results of sampling error analyses. In addition, this plan ranked well in the comparison with other sampling plans which were considered. Consequently, the best-centered sampling plan was chosen for a more complete analysis of data involving areas of 25, 50, 100, 200, and 400 square miles.

As noted previously in the sections on variation of point rainfall with distance and areal representativeness of point rainfall, graphical plots of the data indicated that an equation for average error, E, of the form

$$\text{Log } E = a_3 + b_3 P^{0.5} + c_3 \text{ Log } N \qquad (6)$$

would provide a better fit to the data at high and low values of P than an expression of the form given in (2). Consequently, an expression of the form shown in (6) was applied to data for all five areas. The values of a_3, b_3, and c_3 are tabulated in Table 15.

TABLE 15

REGRESSION CONSTANTS

Area (Sq. Mi.)	Log E a_3	Log E b_3	Log E c_3	Log S a_4	Log S b_4	Log S c_4
25	-1.720	0.65	-0.73	-1.556	0.60	-0.63
50	-1.597	0.56	-0.55	-1.421	0.53	-0.60
100	-1.439	0.59	-0.65	-1.275	0.49	-0.63
200	-1.305	0.49	-0.56	-1.220	0.50	-0.58
400	-1.356	0.54	-0.48	-1.198	0.50	-0.54

More data should be added to the analysis to substantiate the trends in b_3 and c_3 The Water Survey has rain-gage networks in operation which will furnish these data.

Since gage density is a more commonly used variable than the sample size variable, the constants a_3 and c_3 presented in Table 15 were adjusted to express the sample size factor in terms of gage density by the substitution

$$\text{Log } N = \text{Log } A - \text{Log } G \qquad (7)$$

where G is in square miles per gage. Equations for Log E with the adjusted constants are shown in Table 16.

Although the computed E tends generally to increase with area, the individual equations permit

TABLE 16

EQUATIONS FOR AVERAGE AND STANDARD DEVIATIONS

Area (Sq. Mi.)						
	Log E	=	a_5	+ $b_4 P^{0.5}$	+ c_5 Log G	
25		=	-2.740	0.65	0.73	
50		=	-2.531	0.56	0.55	
100		=	-2.739	0.59	0.65	
200		=	-2.594	0.49	0.56	
400		=	-2.605	0.54	0.48	
Combined		=	-2.642	+ $0.794\ A^{-.07}\ P^{0.5}$	+ $0.966\ A^{-.12}$	Log G
	Log s	=	a_6	+ $b_5 P^{0.5}$	+ c_6 Log G	
25		=	-2.395	0.60	0.63	
50		=	-2.440	0.53	0.60	
100		=	-2.535	0.49	0.63	
200		=	-2.555	0.50	0.58	
400		=	-2.603	0.50	0.54	
Combined		=	-2.506	+ $0.692\ A^{-.06}\ P^{0.5}$	+ $0.746\ A^{-.05}$	Log G

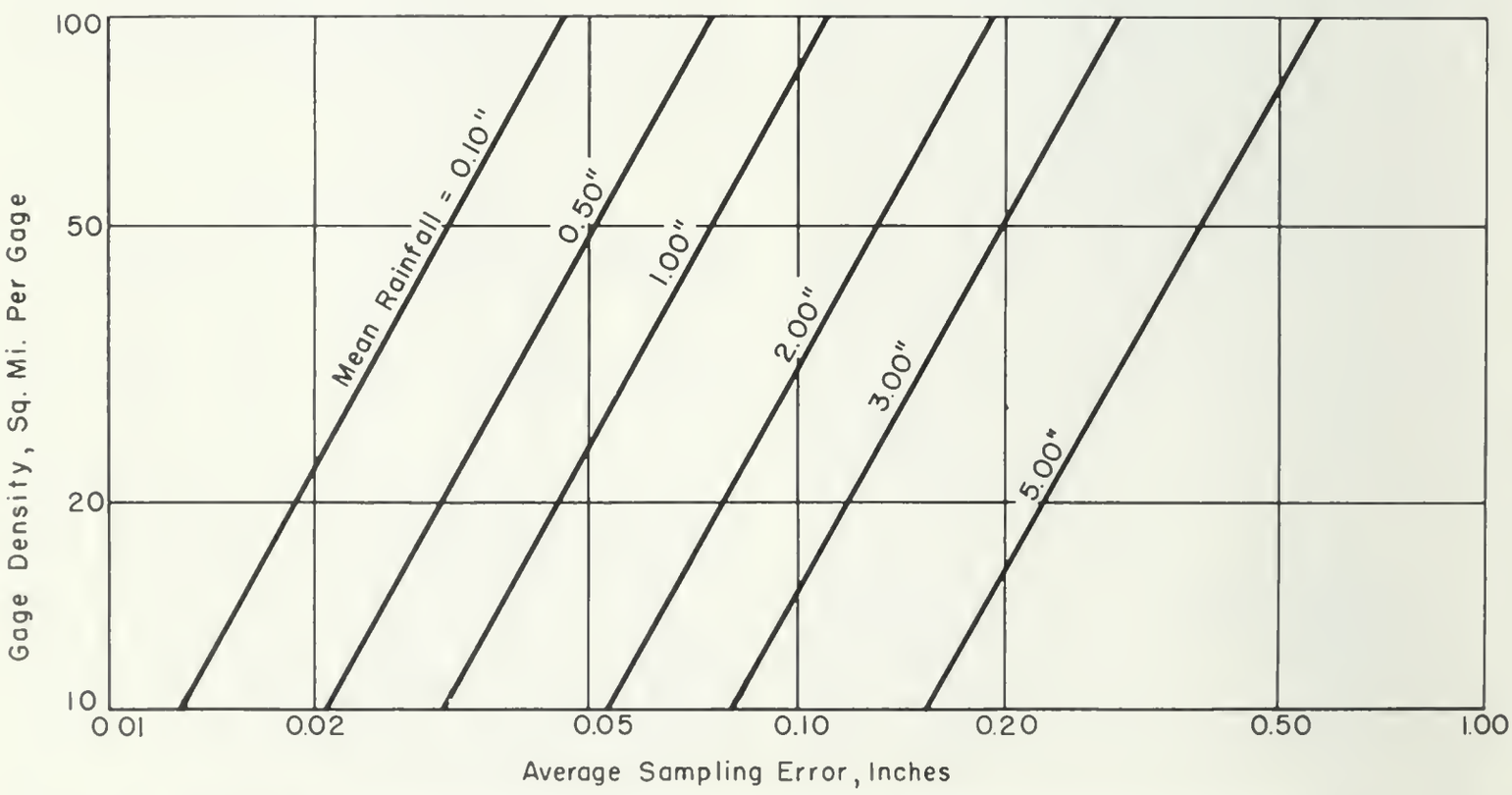


FIGURE 46 RELATION BETWEEN SAMPLING ERROR, STORM SIZE, AND GAGE DENSITY ON 100 SQUARE MILE AREA

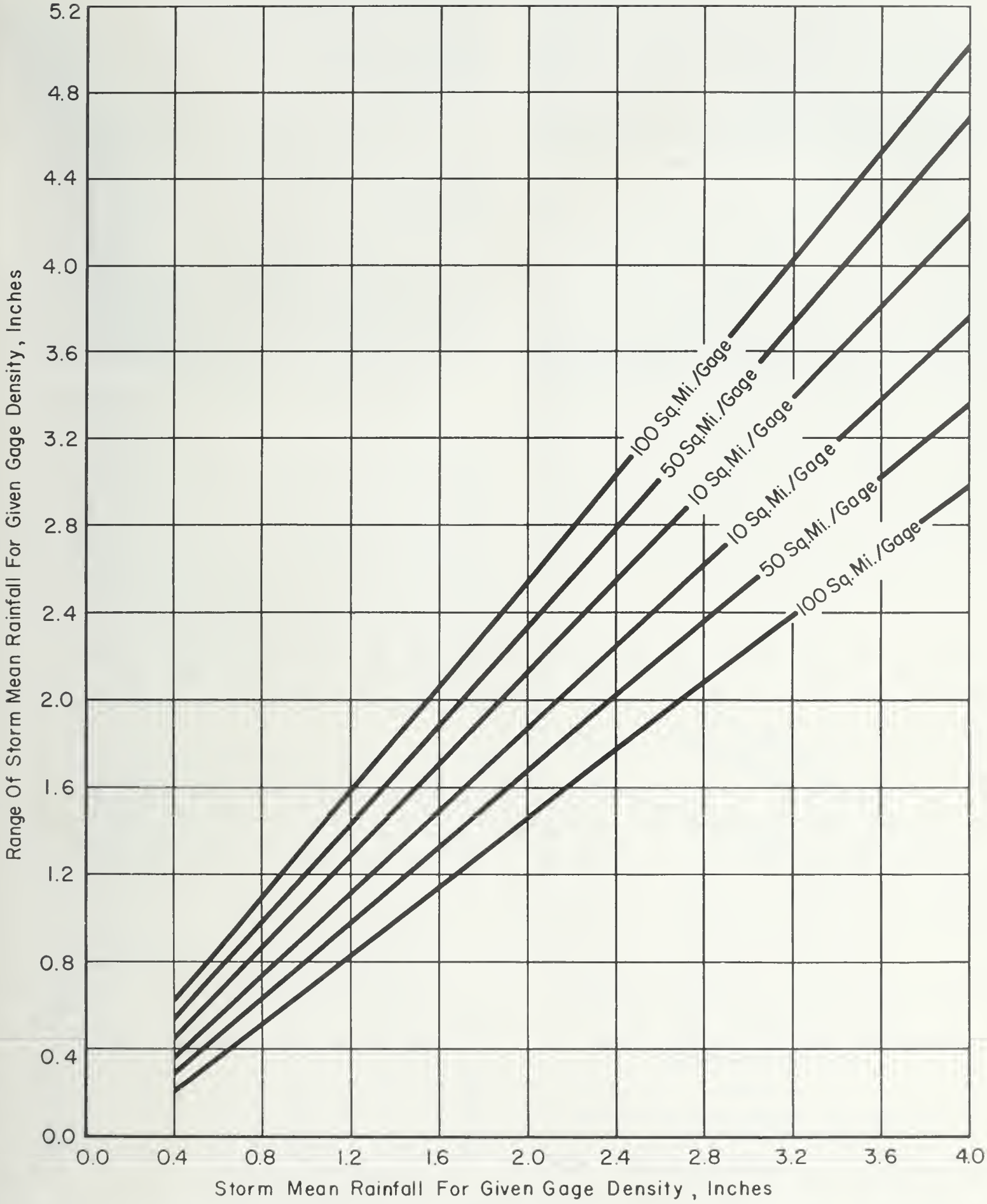


FIGURE 47 95 PER CENT CONFIDENCE RANGE OF STORM MEAN RAINFALL FOR VARIOUS GAGE DENSITIES

random fluctuations in this trend. A combined equation which would represent all five areas was desired for the purpose of smoothing and for interpolating between areas. Since the constants represented by a_5 vary at random, an average of these values may be used in a combined equation. As previously noted, there is a tendency for the values of b_4 and c_5 to decrease in magnitude as the area increases. Consequently, the values of the b_4 and c_5 were expressed as a function of area by expression of the following form:

$$b_4 = k\ A^1 \tag{8}$$

and

$$c_5 = k_1 A^1 \tag{9}$$

The combined equation for Log E which represents all five areas in the analysis is shown in Table 16. Relations for a 100-square mile area obtained from the combined equation are presented in Figure 46.

Confidence Limits. To establish confidence limits, an estimate of the sampling standard deviation was required. This estimate was obtained by replacing Log E in Expression (6) of Log s and fitting the resulting expression to the data, as was done for Log E.

Standard deviations were computed for the same class intervals used in determining Log E. The constant values are shown in Table 15. The values of a_4 and c_4 were adjusted for the gage density factor and the resulting equations for individual areas and for all areas combined are shown in Table 16.

The combined areal equation for Log s can be used to establish confidence limits for areal mean rainfall. The 95 per cent confidence limits for $\bar{P}$ were estimated simply by taking 2s and adding and subtracting from the corresponding value of P. The resulting bands are represented for selected values of G for the 100-square mile area in Figure 47.

EXCESSIVE RAINFALL RELATIONS ON 100-SQUARE MILE AREA

Data collected from 20 recording rain gages during 1950-53 on the 100-square mile Panther Creek watershed (Fig. 3) were used in a limited analysis of excessive rainfall relations. The following excessive rainfall definition of the U. S. Weather Bureau was used in the study

$$R = T + 20 \qquad (1)$$

where R is the rainfall depth in inches and T is the period of observation in minutes. Excessive rates for periods of 30, 60, and 120 minutes were investigated. The above definition gives lower limits of 0.50, 0.80, and 1.40 inches for the 30-, 60-, and 120-minute periods, respectively. In addition to the Panther Creek data, data from 48 gages on the 100-square mile Goose Creek network (Fig. 4) were available for 1953-54, and these have been utilized to a limited extent in the analysis.

Gage Density to Observational Frequency Relations

The effect of gage density upon the frequency of observation of excessive rainfall values in a 100-square mile area was examined first. For this purpose, the Panther Creek network was subdivided into networks of 10, 5, 2, and 1 gages, keeping the distribution in each set as uniform as possible. Similarly, on the Goose Creek area, networks of 24, 12, 6, 3, and 1 gages were selected. The number of excessive amounts recorded by one or more gages for each of the various networks within both areas was then tabulated. Use of special recording rain gages having 12.6-inch diameter orifices and 6-hour charts enabled recording of 15-minute excessive amounts on the Goose Creek area, in addition to those for the 30-, 60-, and 120-minute periods.

Results of this first phase of the study are shown in Table 17. The expected trend for the observational frequency of excessive amounts to increase with increasing gage density is apparent. Of particular interest are the data for 24 and 48 gages in the Goose Creek network. The very slight increase in frequency in going from 24 to 48 gages indicates that the optimum gage density for a 100-square mile area, considering all factors, is probably between 20 and 30 gages. It appears that storms of adequate intensity to provide excessive values for the time periods investigated are of sufficient areal extent and of sufficient duration to be almost always observed by a network of 20 to 30 gages.

TABLE 17

RELATION BETWEEN EXCESSIVE RAINFALL FREQUENCY AND GAGE DENSITY

Number of Gages	Number of Cases for Given Time Period (Min.) 15	30	60	120	Total
Goose Creek Network, 100 Sq. Mi., 1953-54					
1	14	14	5	2	35
3	22	19	14	3	58
6	24	22	14	3	63
12	28	25	18	5	76
24	33	28	21	6	88
48	34	28	22	7	91
Panther Creek, 100 Sq. Mi., 1950-53					
1	--	21	14	5	40
2	--	34	17	10	61
5	--	41	25	14	80
10	--	49	31	15	95
20	--	52	33	18	103

Effect of Gage Density on Observed Maximum

The effect of gage density on the maximum observed excessive amounts within a 100-square mile area was investigated next, using the 1950-53 data for Panther Creek. For this purpose, networks of 20, 10, 5, and 2 gages within the 100 square miles were used. For each storm, a ratio of the maximum amount observed on the network to the amount observed at the central gage in the network was computed. This was done for each network and for 30-, 60-, and 120-minute periods whenever excessive values were recorded by one or more of the 20 gages making up the network of maximum density. Average ratios were then calculated for each of the several networks within the 100-square mile area. The results of this study are summarized in Table 18 where the average ratios for each network within the 100-square mile area are presented for 30-, 60-, and 120-minute periods. The relation is further illustrated in Figure 48.

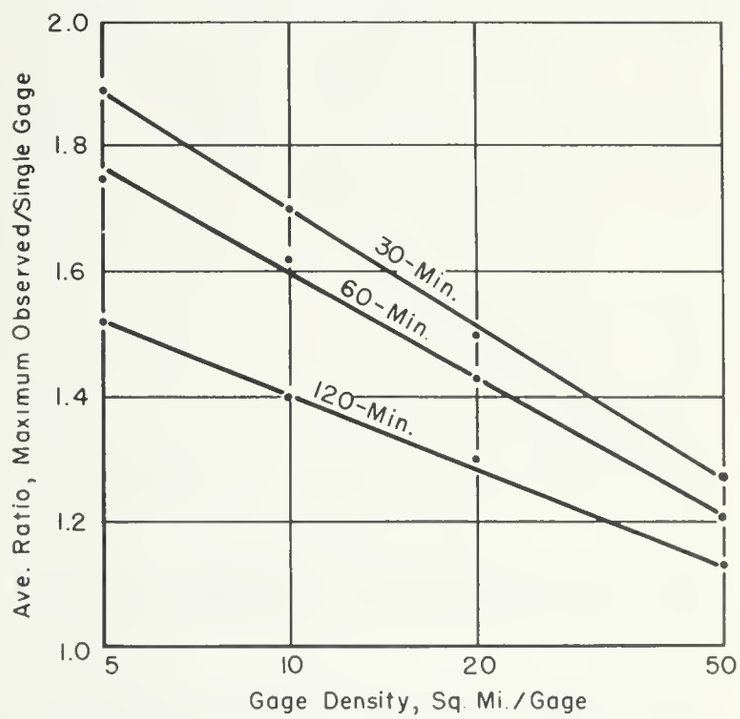


FIGURE 48 EFFECT OF GAGE DENSITY ON MAXIMUM OBSERVED EXCESSIVE RATES

TABLE 18

EFFECT OF GAGE DENSITY ON OBSERVED MAXIMUM AMOUNTS

Number of Gages	Av. Ratio, Maximum Observed/Central Gage Amount for Given Time Periods (Min.) 30	60	120
20	1.89	1.75	1.52
10	1.70	1.62	1.40
5	1.50	1.43	1.30
2	1.27	1.21	1.13

As expected the ratio tends to increase with increasing gage density. The ratio appears to be still increasing at an appreciable rate when the maximum network of 20 gages is reached.

Point-Areal Mean Relations

Next, the relation between point excessive and areal mean excessive amounts was investigated. The areal mean was obtained in each storm by averaging the heaviest amount observed at each of the 20 rain-gage stations for the periods of 30, 60, and 120 minutes. That is, the individual amounts making up the mean may have occurred at different times at the various network stations although all occurred in the same storm period. This definition of the mean was considered more desirable than that obtained by calculating means at a given time during the storm for all gages. The core of heavy rainfall, especially in thunderstorms, frequently covers a relatively small area; consequently, the heaviest storm rates occur at different times in moving across an area of 100 square miles. Since such factors as soil erosion and sedimentation appear to be closely related to the short-period, heavy rainfall rates experienced in storms, the method used for calculating the means was considered to have the most practical application.

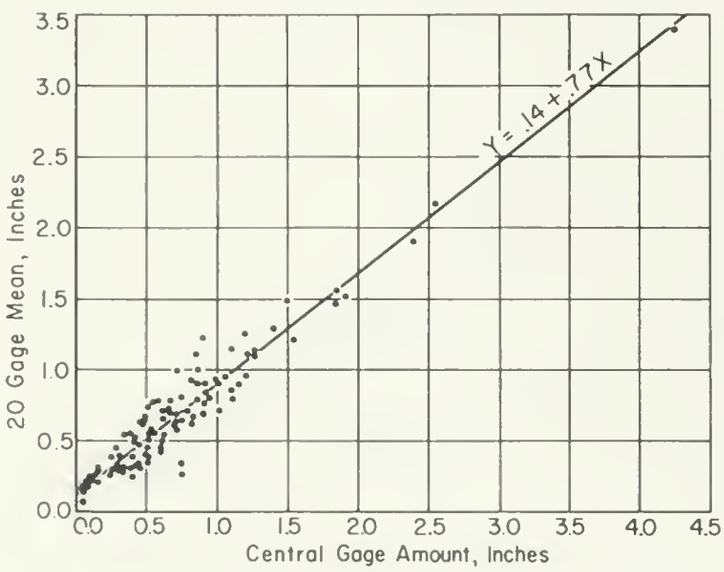


FIGURE 49 RELATION BETWEEN POINT AND AREAL MEAN RATES UNDER EXCESSIVE RATE CONDITIONS

Figure 49 illustrates the relations between the point rainfall observed at the central gage and areal mean rainfall for 30-, 60-, and 120-minute periods. The three periods were combined into one relation after separate graphical plots indicated insignificant differences among the relations for individual periods. Points in Figure 49 include all cases in which an excessive amount was recorded by one or more gages in the 20-gage network. The regression equation indicates that the point rainfall observed at the center of a 100-square mile area is slightly above the areal mean rainfall when excessive amounts are experienced (above 0.50 inch). This trend is to be expected when storm movement is considered. For example, assume two thunderstorms of the same size and same intensity, one passing over the network with its core passing through the center of the network, while the core of the other storm passes along or near the border of the network. Obviously, the heaviest rates will be observed at the central gage when a given storm passes through or near the center of the network and these storms will also provide the most frequent areal mean excessive rates. A correlation coefficient of 0.96 was obtained between the 20-gage mean and the central-gage point rainfall.

Observations used in the preceding analysis of point-areal mean relations included a considerable number in which neither the central-gage amount nor the areal mean rainfall represented an excessive amount, since only one excessive value among the 20 gages was required for inclusion in the analysis. To determine whether the inclusion of values below the excessive level materially affected the point-areal relation, another analysis was made in which only those cases with an excessive amount at the central gage were included. This analysis resulted in development of a regression equation given by

$$Y = 0.11 + 0.78\ X \qquad (2)$$

where Y is the areal mean rainfall in inches and X is the central gage amount in inches. This equation is insignificantly different from the one illustrated in Figure 49. A correlation coefficient of 0.97 was obtained compared to 0.96 for the curve in Figure 49. Results indicate that Figure 49 can be used for estimating the areal mean maximum or areal mean excessive rainfall on a 100-square mile basin for short periods within storms, when a point rainfall observation is available at or near the center of the area.

A study of the data indicated the central gage gives a very good measure of the frequency of excessive areal mean rainfall. For example, on Panther Creek during 1950-53 the number of areal mean excessive values was practically the same as the number observed at the central gage. For a 30-minute period, 21 excessive amounts were observed on both a point and areal basis. For a 60-minute period there were 14 point excessive amounts and 13 areal excessive amounts, while for a 120-minute period there were 5 excessive values for both cases. Combining all three periods, there were 40 excessive amounts observed at the central gage compared to 39 cases of excessive areal mean rainfall. However, this does not mean that the point and areal mean rates all occurred in the same storms. By comparison, it was found that excessive areal means occurred 17 out of the 21 times that excessive amounts were observed at the central gage. Similarly, 11 out of 14 point excessive amounts occurred in conjunction with excessive areal mean values for the 60-minute period, while all five of the point and areal excessive amounts coincided for the 120-minute period.

Figure 50 is an illustration of the occurrence of excessive rainfall amounts over the network during an exceptionally heavy storm during the evening of June 8, 1951. Heavy excessive amounts were observed at all stations within the network

during the storm period. An areal average of 7.21 inches fell during the entire storm which lasted about four hours. Figure 51 illustrates the distribution of excessive rainfall in the area by means of area-depth curves. It is interesting to note that the square-root relation for total storm rainfall discussed in the section on area-depth relations provides a good fit in these cases of unusually heavy short-period amounts within a storm.

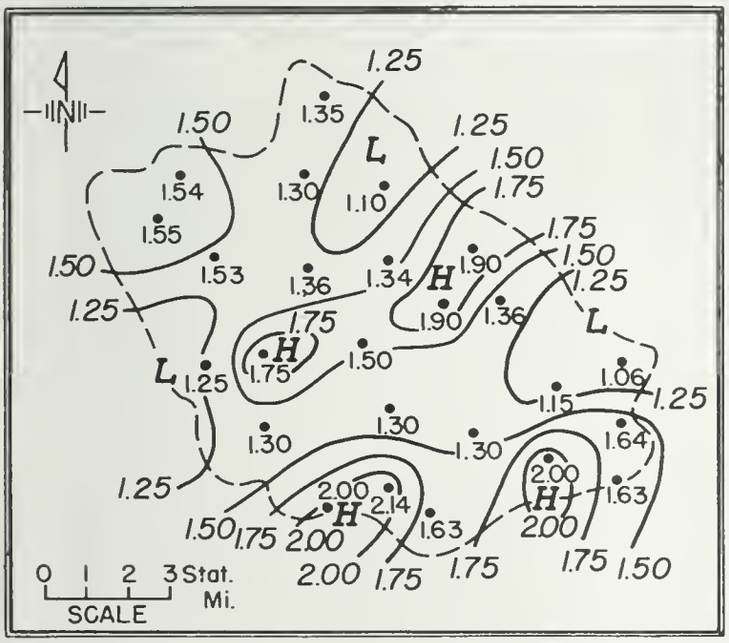


FIGURE 50 30-MINUTE EXCESSIVE RATES, JULY 8, 1951

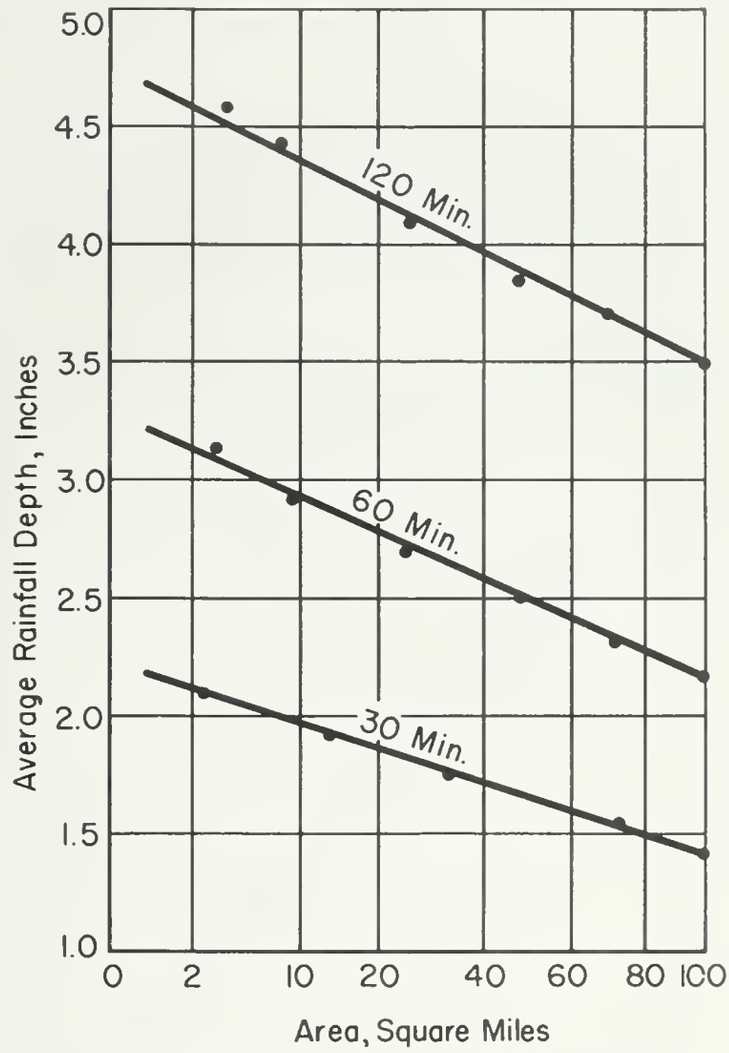


FIGURE 51 EXCESSIVE RATE AREA-DEPTH RELATIONS, JULY 8, 1951

RELATION BETWEEN POINT AND AREAL RAINFALL FREQUENCIES

Although much information has been published on point rainfall frequencies, little data are presently available in the literature on areal rainfall frequencies. The following presents limited information on the subject, obtained from a 20-gage network on the 100-square mile Panther Creek watershed during the period 1948-53.

Using data from this network, comparisons have been made between point and areal mean rainfall frequencies for the above 6-year period for areas of 25 and 50 square miles within the 100-square mile network. Because of minor changes in some of the gage locations from year to year, it was not possible to obtain a 6-year comparison for the 100-square mile area. However, data were available for a 3-year comparison and these results are also given.

Within each area, the point rainfall frequencies were obtained from the rain gage in the center of the area. Areal mean rainfall was calculated from

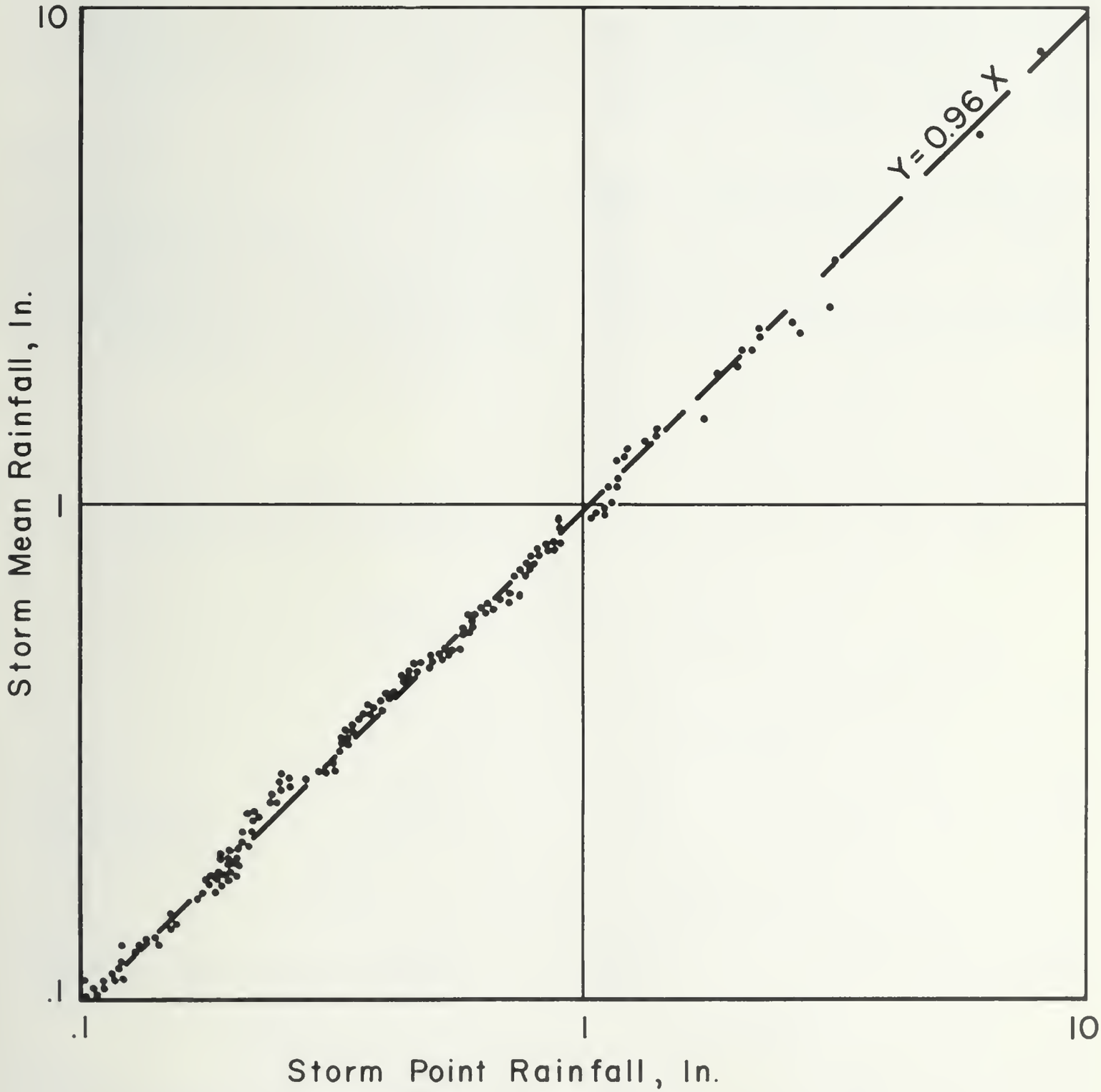


FIGURE 52 COMPARISON BETWEEN POINT AND AREAL MEAN RAINFALL AT EQUAL FREQUENCIES ON 25 SQUARE MILE AREA

the arithmetical average of all gage observations. Within the 25- and 50-square mile areas studied, there were 6 and 12 gages, respectively. To obtain comparisons between point and areal frequencies, all storms for each area were ranked by mean rainfall. Similarly, storm point rainfall totals at the center of each area were ranked. The rankings then provided a frequency distribution of point and areal mean rainfall within a 6-year period for the 25- and 50-square mile areas and within a 3-year period for the 100-square mile area. Regression equations and correlation coefficients were obtained between point and areal mean rainfall frequencies for each area, treating equal positions of rank as pairs of observations. To illustrate the pairing procedure, the highest point rainfall observed at the center of the 25-square mile area, 8.22 inches, was paired with the highest mean rainfall for this area during the 6-year comparison period, which was 8.16 inches. Obviously, the paired observations in many cases occurred in different storms, since the data were paired on a frequency distribution basis.

Relations were developed for storm rainfall, in which a single storm was defined as all rainfall unseparated by a break in precipitation of six hours or more. Consequently, several showers were frequently included in a storm as defined in this study. The results are perhaps also indicative of daily rainfall. The study was confined to shower-type rainfall, the type of storm from which flash floods on small areas occur. The results are considered indicative and not definitive of point-areal frequency relations.

The regression equations obtained for the 25- and 50-square mile areas were $Y = 0.96X$ and $Y = 0.94X$, respectively. The 6-year point-areal comparison for the 25-square mile area is illustrated in Figure 52.

Results of the study indicate that an excellent relationship exists between point and areal rainfall frequencies. A correlation coefficient of 0.99 was obtained between point and areal rainfall for both the 25- and 50-square mile areas. For the 100-square mile area, a correlation coefficient of 0.99 was obtained and the regression equation was $Y = 0.97X$, very close to the equations obtained for the 25- and 50-square mile areas. Indications are that areal mean rainfall frequencies are close to, but slightly less than, equivalent point rainfall frequencies for small areas.

A 3-year comparison for another set of 25- and 50-square mile areas within the 100-square mile watershed produced regressions of $Y = 0.97X$ for both areas, agreeing closely with the 6-year comparisons. A 3-year frequency comparison was also made using data only for excessive rainfall rates, based on the definition of excessive rainfall given in the previous section of this bulletin. Combining data for excessive 30- and 60-minute amounts, a regression equation, $Y = 0.92X$, was obtained.

A MICROMETEOROLOGICAL STUDY OF RAINFALL VARIABILITY

Introduction

Analyses of rainfall data from several concentrated rain-gage networks in central Illinois during recent years have revealed the frequent presence of rainfall gradients exceeding 0.25 inch per mile in shower-type rainfall. The relation between rainfall variability and either distance or area is pertinent in the determination of gage density requirements for specific purposes, such as: the collection of small scale climatological data, hydrologic projects involving runoff and sedimentation investigations, agricultural research, and studies concerned with establishing the utility of radar for quantitative precipitation measurements. Also, the magnitude of rainfall variability with distance should be considered in determining the sensitivity and calibration accuracy required for recording rain gages, especially those employed in routine observational programs where concentrated networks are not feasible.

As part of a program to investigate the quantitative relations existing between rainfall variability and distance, a micro-network of rain gages was established in 1953. Since the greatest variation in rainfall with distance normally occurs with shower-type rainfall, data were collected during the spring, summer and fall seasons of 1953-54. Similar observations on a mesometeorological scale were available for comparison purposes from other networks described in the section on rain-gaging facilities.

Description of Network

A micro-network of 18 gages was established on level meadow land at the University of Illinois Airport. Its design was based upon available information concerning rainfall gradients obtained from rainfall studies on several concentrated watershed networks during 1948-52. The network consisted of pairs of 8-inch non-recording gages spaced six feet apart at each of nine stations which were located at intervals of 300 feet to form a square grid pattern (Fig. 8). Information on storm duration and rainfall intensity was obtained from a recording rain gage installed adjacent to the micro-network. Wind data were obtained from an Aerovane system located about 0.5 mile from the network.

The calibration of each gage was checked before installation. The level of each gage was checked periodically and the gages kept as free of dirt, insects and other foreign material as possible. Measurements of rainfall were made for each occurrence of precipitation. These measurements were made as soon as possible after the end of each storm to minimize evaporation effects. Careful attention was given to gage exposures, maintenance, and observational techniques. To minimize the reading error, experienced observers read the rain-gage amounts to the nearest .001 inch.

Analysis of Data

Data were collected from 93 storms from March to November, during 1953 and 1954. For each storm, average precipitation differences were calculated for the various sets of gages located 6, 300, and 600 feet apart on the grid pattern. Results of this analysis, classed by storm size, are shown in Table 19. The mean maximum differences shown in Table 19 are the averages for the highest difference observed in each storm. The absolute maximum is the highest reading obtained during the 1953-54 test period for each class of storm size, as measured by the network mean rainfall.

TABLE 19

DIFFERENCES BETWEEN GAGE PAIRS

	Differences (in.) for Given Storm Sizes (in.)			
	0.01-0.19	0.20-0.49	0.50-0.99	1.00-1.99
		6-Foot Pairs		
Average	0.003	0.005	0.009	0.020
Mean Maximum	0.009	0.013	0.022	0.047
Absolute Maximum	0.015	0.046	0.041	0.110
Number of Storms	46	19	15	13
		300-Foot Pairs		
Average	0.005	0.009	0.013	0.027
Mean Maximum	0.013	0.025	0.034	0.079
Absolute Maximum	0.026	0.049	0.048	0.165
Number of Storms	46	19	15	13
		600-Foot Pairs		
Average	0.006	0.012	0.016	0.038
Mean Maximum	0.015	0.027	0.034	0.087
Absolute Maximum	0.034	0.047	0.070	0.231
Number of Storms	46	19	15	13

Reference to Table 19 shows that average 6-foot differences for storms having mean rainfall up to 0.50 inch are rather insignificant. However, appreciable differences were usually observed with the heavier storms and occasionally with light storms. Since the differences have a definite tendency to increase with storm size, they cannot be attributed solely to reading error. Another factor which may have contributed to the differences observed between the 6-foot pairs is the turbulence created in the wind flow about the gages. There was no way in which to separate the effects of reading and turbulence errors from the real differences occurring in the surface rainfall pattern. Considering the careful attention which was given to gage exposures, gage maintenance, and observational techniques, it is believed that the differences which were obtained between the 6-foot pairs represent the minimum to be expected in shower-type rainfall. For the 6-foot pairs a correlation coefficient of 0.79 was obtained between average storm differences and storm size.

The trend for the average difference to increase with increasing storm size and with distance is further illustrated by the summarized data for the 300-foot and 600-foot pairs in Table 19. A correlation coefficient of 0.80 was obtained between average difference and mean rainfall for the 300-foot pairs. Similarly, a correlation coefficient of 0.72 was obtained with the 600-foot pairs.

The data were further examined for differences between 6-foot gage pairs arising from meteorological factors other than mean rainfall. These factors include storm duration, mean rainfall rate, and wind. Results of this phase of the study are presented in Table 20, where the relative variability of the average differences has been related to each meteorological factor. The variability factor was obtained for each storm by dividing the average difference for pairs at a given distance by the areal mean rainfall and multiplying by 100 to convert the factor to per cent.

TABLE 20

RELATIVE VARIABILITY TRENDS FOR 6-FOOT GAGE PAIRS

Av. Variability (%) for Given Size of Event

Mean Rainfall, in.				
0.01-0.09	0.10-0.19	0.20-0.49	0.50-0.99	1.00-1.99
6.1	2.5	1.9	1.4	1.3
Storm Durations, hrs.				
0.1-2.0	2.1-4.0	4.1-6.0	6.1-12.0	12.1-24.0
5.8	1.9	2.1	1.9	2.0
Mean Rainfall Rate, in/hr				
0.01-0.05	0.06-0.10	0.11-0.20	0.21-0.30	Over 0.30
5.8	2.7	2.3	2.2	1.8
Wind Speed, mph				
0-5	6-10	11-15	16-20	21-30
3.6	3.7	2.8	2.8	2.4

Examination of Table 20 shows that the relative variability tends to decrease with increasing mean rainfall, this trend being more pronounced for the light storms. A similar trend was observed for storm duration up to four hours, after which, there appeared to be no significant change in the average relative variability with increasing storm duration. Mean rainfall rate produced its greatest effect upon the relative variability at low values. A slight trend was observed for the relative variability to decrease with increasing magnitude of wind speed, the average relative variability tending to level off with high values of wind speed.

Because of the magnitude of the computations involved, no attempt was made to combine all the possible variables into a multiple regression equation, especially since a relatively high correlation was obtained between the various differences and storm size of mean rainfall. Scatter diagrams relating the 6-foot relative variability to mean rainfall, wind speed, storm duration, and mean rainfall rate are illustrated in Figure 53.

For comparison with the small scale differences in Table 19, some results from the study discussed in the section on the variation of point rainfall with distance are summarized in Table 21. The relations shown in this table were obtained from observations in shower-type rainfall collected on the Panther Creek and Goose Creek networks shown in Figures 3 and 4.

The magnitude of the variability between 6-foot 300-foot and 600-foot pairs on a monthly and seasonal basis is illustrated in Table 22. A trend for average differences between gage pairs to increase with distance and with areal mean rainfall was found for both monthly and seasonal precipitation. This trend was less pronounced than that found between average differences and storm mean rainfall. Correlation coefficients of 0.60, 0.58, and 0.48 were obtained between monthly rainfall and average differences of 6-foot, 300-foot and 600-foot pairs, respectively. Although a maximum monthly difference of 0.27 inch was observed between 600-foot pairs on one occasion, average monthly differences were generally less than 0.05 inch.

Computations were made of the accuracy with which the central gage within the grid pattern measured the storm mean rainfall for the entire area. Results are summarized in Table 23 where the average, standard and maximum errors have been related to storm size. Mean rainfall was obtained from the arithmetical average of all gages. The errors were found to increase with increasing storm size, although remaining relatively small.

TABLE 21

VARIATION OF POINT RAINFALL WITH DISTANCE

Storm Size (in.)	Av. Difference (in.) at Given Distance	
At Given Point	1 Mile	3 Miles
0.10	0.06	0.09
0.25	0.08	0.11
0.50	0.10	0.14
1.00	0.14	0.20
1.50	0.18	0.26
2.00	0.23	0.33

TABLE 22

MONTHLY AND SEASONAL DIFFERENCES BETWEEN GAGE PAIRS

	Mean	Difference (in.) for Given Distance					
		6-Ft		300-Ft		600-Ft	
1953	Rainfall, in.	Av.	Max.	Av.	Max.	Av.	Max.
April	1.30	0.022	0.067	0.023	0.054	0.032	0.094
May	1.53	0.022	0.047	0.028	0.078	0.028	0.059
June	3.39	0.016	0.034	0.032	0.100	0.039	0.079
July	3.15	0.036	0.057	0.032	0.080	0.036	0.066
Aug.	0.59	0.012	0.032	0.016	0.036	0.021	0.040
Sept.	0.41	0.006	0.016	0.008	0.031	0.014	0.039
Oct.	1.74	0.028	0.061	0.039	0.120	0.055	0.123
1954							
March	1.18	0.013	0.034	0.023	0.060	0.023	0.044
April	4.61	0.043	0.083	0.083	0.234	0.145	0.270
May	4.34	0.026	0.053	0.025	0.081	0.029	0.074
June	2.59	0.050	0.112	0.044	0.122	0.055	0.102
July	2.79	0.020	0.036	0.037	0.090	0.054	0.125
Aug.	4.51	0.028	0.055	0.027	0.088	0.025	0.083
Sept.	0.30	0.007	0.014	0.013	0.032	0.025	0.036
Oct.	4.51	0.032	0.057	0.028	0.076	0.044	0.110
1953							
Apr.-Oct.	12.11	0.053	0.092	0.102	0.234	0.167	0.327
1954							
Mar.-Oct.	24.83	0.113	0.219	0.175	0.429	0.200	0.482

TABLE 23

RELATION BETWEEN AREAL MEAN RAINFALL AND CENTRAL GAGE POINT RAINFALL

	Storm Mean Rainfall, in.				
	0.01-0.09	0.10-0.19	0.20-0.49	0.50-0.99	1.00-1.99
Average Error	0.003	0.003	0.005	0.007	0.013
Standard Error	0.004	0.004	0.007	0.009	0.017
Maximum Error	0.013	0.011	0.017	0.025	0.044
Number of Storms	28	18	19	15	13

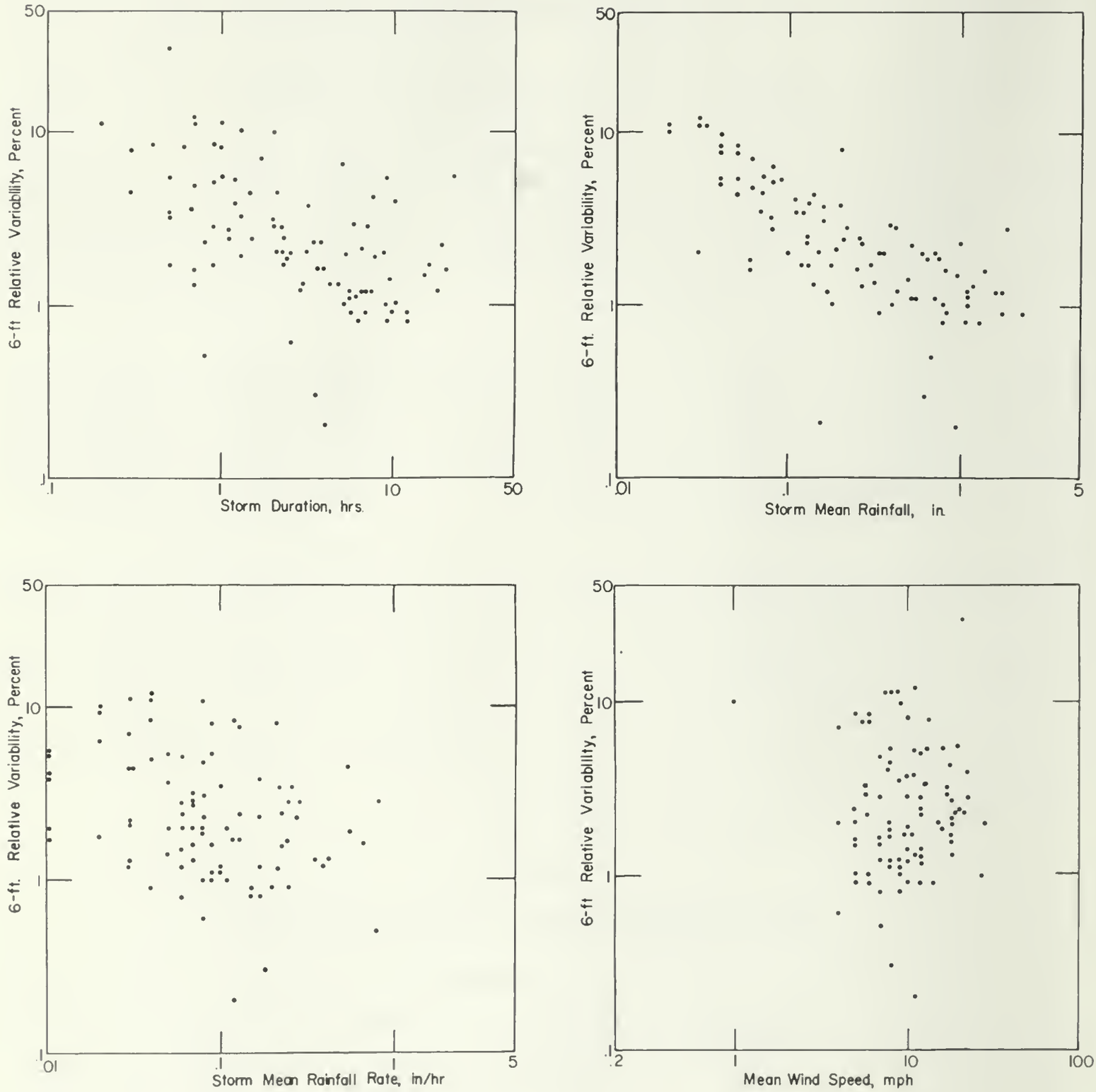


FIGURE 53 EFFECT OF SEVERAL METEOROLOGICAL FACTORS ON RELATIVE VARIABILITY OF STORM RAINFALL

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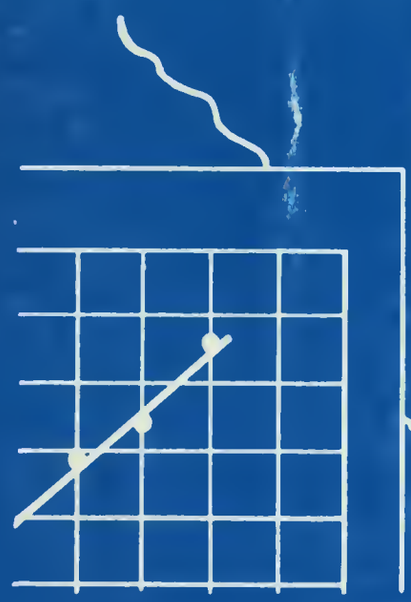
Frequency Relations for Storm Rainfall in Illinois

by F. A. HUFF and J. C. NEILL



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Frequency Relations for Storm Rainfall in Illinois

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Rainfall in Illinois

by F. A. HUFF and J. C. NEILL

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FREQUENCY RELATIONS FOR STORM RAINFALL IN ILLINOIS

by F. A. Huff and J. C. Neill

SUMMARY AND CONCLUSIONS

If the physical laws governing the inception and distribution of precipitation were completely understood, it would be unnecessary to resort to many of the statistical approaches now used by meteorologists and hydrologists to predict the future distribution of storm rainfall. However, even the basic physical processes involved in the production of precipitation in the atmosphere have not been adequately defined or evaluated at present, although considerable research is being directed toward the solution of this problem. Consequently, the investigator of precipitation frequency relations is forced to depend primarily upon application of statistical methodology to samples of observational data which he hopes are representative of the population distribution.

The production and distribution of precipitation are obviously dependent upon complex reactions in nature. Therefore, the authors of this report could find no firm basis for selecting in advance any one of the several commonly used statistical distributions, as best for the analysis of Illinois frequency data. It did not seem logical to pass judgment when so little is known about the basic laws and processes governing rainfall distribution. Rather, it appeared that the selection of a statistical technique should be determined from the goodness-of-fit of raw data to each of several statistical distributions, which appeared as promising yardsticks for estimating future events. Consequently, this approach was followed throughout the investigation in establishing annual and seasonal frequency relations.

Complete objectivity in establishing frequency relations was not possible, since tests of several statistical methods showed that none was distinctly superior in fitting the data samples. Consequently, in the final analysis, the statistical distributions selected for computing annual and seasonal relations were based to some extent upon other available meteorological and climatological information, as discussed in the text. The results represent frequency estimates based upon analysis and evaluation of all information available to the authors during the investigation.

The following conclusions or observations resulted from analyses accomplished in various phases of the investigation of frequency relations.

1. Tests of several statistical distributions commonly used or recommended for frequency analysis of rainfall showed that no one method was distinctly superior in fitting the raw data.
2. Comparison of the methods of moments and least squares in fitting raw data to a given distribution indicated that differences obtained are relatively small and not significant from a practical standpoint.
3. Evaluation of frequency relations developed on both a station and areal basis led to the conclusion that such relations are most reliable when determined for geographical areas. In addition to providing better estimates of average relations, the areal method provides a means for determing various probability levels at a point for a given return period. Also, the range of precipitation to be expected within areas of approximately homogeneous precipitation climate can be calculated for given return periods.
4. A comparison was made between frequency relations derived from several statistical distributions and from graphical analysis. Results indicated that simple graphical methods are generally satisfactory for the analysis of frequency data, provided that only a mean frequency curve is required. However, mathematical procedures should be applied when a definition of the reliability of the average relation is needed.
5. Comparison of sectional frequency relations based on data for periods of 10, 20, and 40 years indicated that little difference exists in such relations when the observational period is 20 years or longer.
6. Analysis using data for the partial duration series indicated an excellent fit of areal data when the logarithms of precipitation were related to the logarithms of recurrence intervals.
7. A close association between large-scale and small-scale climatic events was indicated by the magnitude of annual maxima observed during drought and heavy precipitation periods, and by the correlation between annual and storm precipitation.

8. The year-to-year variability of heavy storms is greatest in summer throughout Illinois, with little difference in the time variability in the other three seasons.
9. Analysis of the monthly and seasonal distribution of annual maxima indicated that summer (June-August) is the season of maximum frequency of heavy storms in northern and north central Illinois. In south central Illinois there is little difference among the number of occurrences in summer, spring, and fall; while in the extreme southern part of the state, spring is the season of maximum frequency. As would be expected, winter is the season of minimum frequency throughout the state, but this minimum becomes less pronounced is moving from north to south through the state and differs little from the summer frequency in extreme southern Illinois. The annual maxima occurrences are quite evenly distributed throughout the four seasons in extreme southern Illinois. However, the unusually severe storms which produce the long return period values are most likely to occur in summer over the entire state.

ACKNOWLEDGMENTS

This report was prepared under the general direction of William C. Ackermann, Chief of the Illinois State Water Survey, and the guidance of Glenn E. Stout, Head of the Meteorology Section.

The authors are indebted to Dr. Ven Te Chow, Professor of Hydraulic Engineering, and Dr. H. W. Norton, Professor of Agricultural Statistical Design and Analysis of the University of Illinois, for consultation and suggestions on several aspects of the investigation. The authors are also indebted to Dr. Chow for his review of this manuscript.

Certain phases of the analyses were greatly expedited through use of facilities at the Statistical Service Unit and the Digital Computor Laboratory at the University of Illinois.

Within the Illinois State Water Survey, credit is due Stanley A. Changnon, Jr., for supervision of the punch card work and drafting of illustrations.

INTRODUCTION

Purpose

Considerable information on precipitation frequencies for periods of 5 minutes to 24 hours has been provided by Yarnell [1] and the U. S. Weather Bureau.[2] However, longer periods of precipitation which are of importance in agriculture, water supply replenishment, and hydrologic design have not been analyzed in detail for Illinois. Consequently, analyses were performed to determine the recurrence interval of heavy precipitation encompassing periods of 1 to 10 days. An early phase of the study was committed to a comparison of several commonly used approaches to frequency analysis of maximum data, in an effort to determine the most representative statistical distribution for use with the Illinois data.

Scope

To accomplish a detailed analysis of heavy precipitation frequencies within the state, annual and seasonal maximum precipitation data were used from 39 stations for the 40-year period 1916-55. Calendar-day data were also utilized in order to include U. S. Weather Bureau cooperative station data,[3] thus providing a relatively large number of stations for determining frequency relations in Illinois. Stations were chosen to give a distribution as uniform as possible throughout the state.

Illinois was divided into four sections of generally similar precipitation climate. For each of these sections, average frequency relations were developed for precipitation periods of 1 to 10 days on both an annual and seasonal basis. In this report, winter includes December through February, and spring encompasses March through May. June through August, and September through November are considered as summer and fall, respectively.

For comparison purposes, an analysis was accomplished on 1-day precipitation data based upon the partial duration series approach. Empirical transformation factors for changing from annual maximum precipitation to partial duration frequencies and from clock-hour maximum to maximum precipitation values were determined for the absolute storm periods of 1 to 10 days. Studies were also made of the actual duration in hours of precipitation during storm periods of 1 to 10 days, the relation between storm and annual precipitation, and the monthly and seasonal distribution of heavy storms.

General Approach

It was decided that the most reliable frequency relations would be obtained by determining a relation to represent the average within an area and then defining the reliability of this average areal relation. Standard errors were used to indicate the range of values to be expected within each area and thus define the reliability of values chosen from the average curve. Although the study was devoted mainly to determining average frequency relations for sections of Illinois, frequency relations at individual stations were also investigated to ascertain the representativeness of these data for defining frequency relations within the surrounding area.

Obviously, areas chosen for determining average relations should have a homogeneous precipitation climate. However, to obtain areas of sufficient size which would contain enough stations to define reliably the average frequency relation, the authors found it necessary to deal with areas having approximately the same climate.

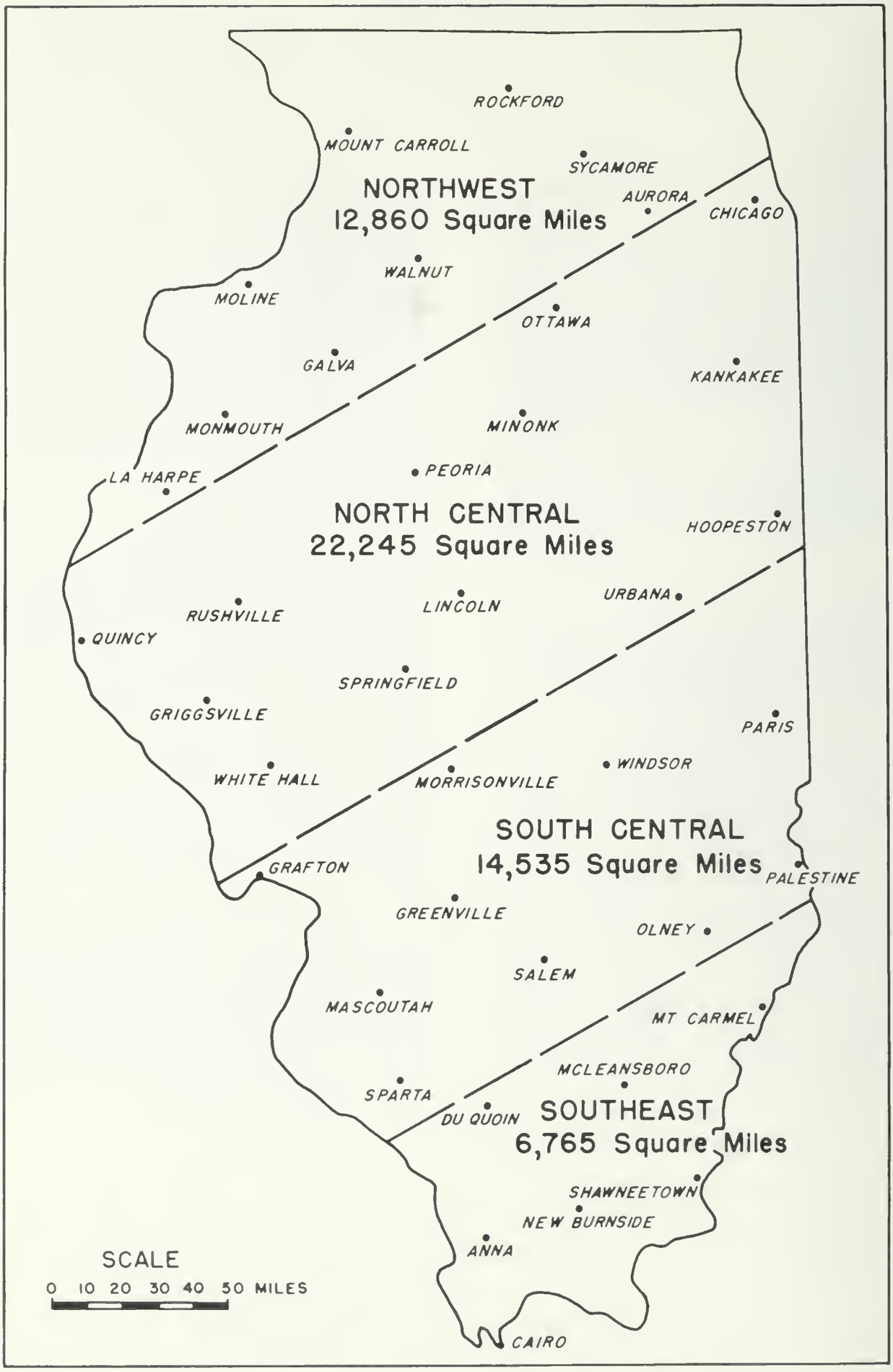

FIGURE 1 SECTION AND STATION LOCATIONS

A number of delineations of sections were tested before the final ones were selected. Station locations, section boundaries and section sizes are shown in Figure 1. The sections were selected on the basis of means, standard deviations, and coefficients of variations of the annual maximum precipitation for the individual stations and on consideration of meteorologic and climatic factors involved. Thus, while the selections were partially subjective, it is believed that the grouping is generally representative of storm rainfall climate within these areas of Illinois. Other groupings tested included a 3-section latitudinal division corresponding to that used by the U. S. Weather Bureau in its state climatological summaries prior to 1957, a 9-section division corresponding to the present climatic sections within the state used by the Weather Bureau, a 3-section longitudinal division, a 4-section latitudinal division, a 4-section division from northwest to southeast throughout the state, and a 4-section division from southwest to northeast.

In addition, an evaluation was made of several statistical approaches to determine if a distinctly preferable analysis technique existed among several commonly used. Frequency curves were fitted to the data using the log-normal distribution and the methods of Gumbel,[4] Jenkinson,[5] Chow,[6] and Frechet.[7] The characteristics of the several methods tested are discussed in another section. For comparative purposes, curves were determined from the raw data by the method of least squares and by the method of moments, except for the Frechet method. In the case of the least squares method of curve fitting, the correlation coefficient and the standard error provide an objective means of comparing the goodness-of-fit for the various frequency distributions.

To help define the range of individual point values about the areal average, standard errors were determined at various recurrence intervals along each regression curve. These standard errors were used in conjunction with corresponding means to determine probabilities covering the range from 10 to 90 percent. Thus, at any given recurrence interval, average rainfall values may be determined from the average curve for the section, along with the expected range of precipitation values for selected probabilities.

Use of average relations for each section creates a discontinuity at the boundary between sections. For example, as one moves from the northern edge of the Southeast Section to the southern edge of the South Central Section (Fig. 1) the frequency relation may abruptly change. The authors feel that this is a minor deficiency which is more than offset by the greater reliability of frequency relations determined by average areal relationships. The average frequency relations could have been used in conjunction with the individual station data to provide isocontour maps for the state for selected recurrence intervals. This may be achieved by using the slope of the areal average curve in conjunction with the mean of the annual maximum precipitation for each station to obtain an adjusted frequency curve for each station. The adjusted frequency curves may then be used to obtain isohyetal maps. The results of this procedure are illustrated in Figure 2, which

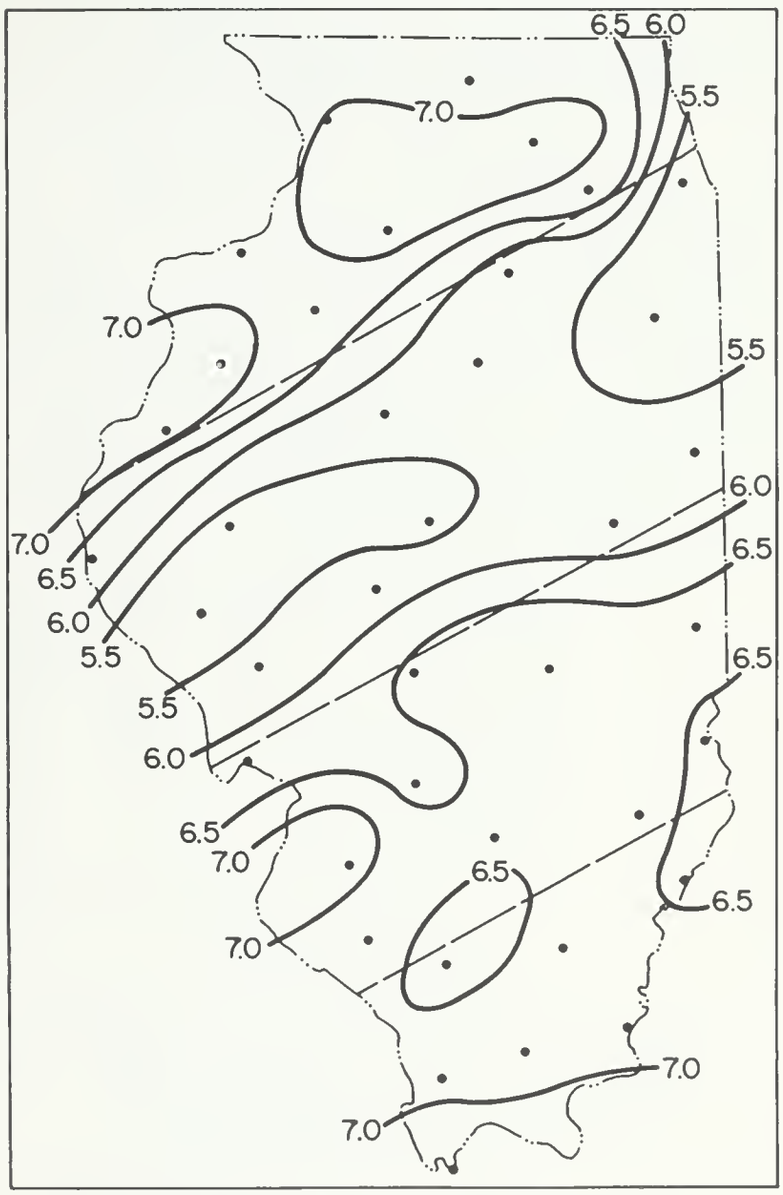

FIGURE 2 50-YEAR RECURRENCE OF 1-DAY PRECIPITATION USING FRECHET METHOD AND DISTRIBUTION INDEX

is a 50-year frequency map for 1-day precipitation obtained by fitting the raw data to the Frechet[7] distribution by the method of least squares. This technique provides a smoothed isohyetal pattern incorporating an area distribution index in the form of the average curve slope. However, the isohyets

derived from this technique may still be misleading, since even the means of the annual maximum precipitation at stations are considerably affected by extreme values within the sampling period. For comparison, an unsmoothed 50-year frequency map obtained from individual station curves using the Frechet method is presented in Figure 3.

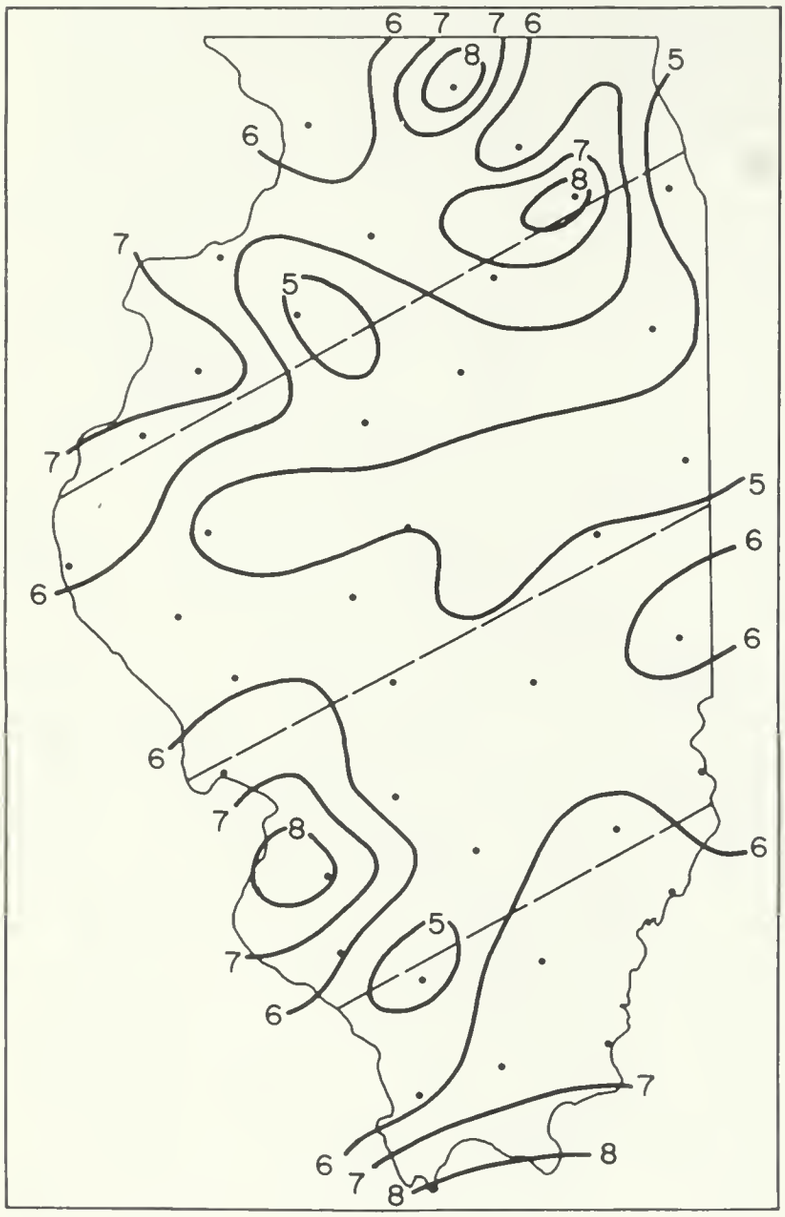

FIGURE 3 50-YEAR RECURRENCE OF 1-DAY PRECIPITATION USING FRECHET METHOD WITHOUT SMOOTHING TECHNIQUE

A simpler cartographic method would assume the section mean values to represent conditions at the central point in each section. An isohyetal map may then be obtained by drawing isohyets parallel to the section boundaries, using linear interpolation between section centers to establish isohyetal values.

As mentioned earlier, the authors prefer to present the study results in the form of areal averages which are believed to provide more realistic values. The sensitivity of isohyetal patterns to sampling

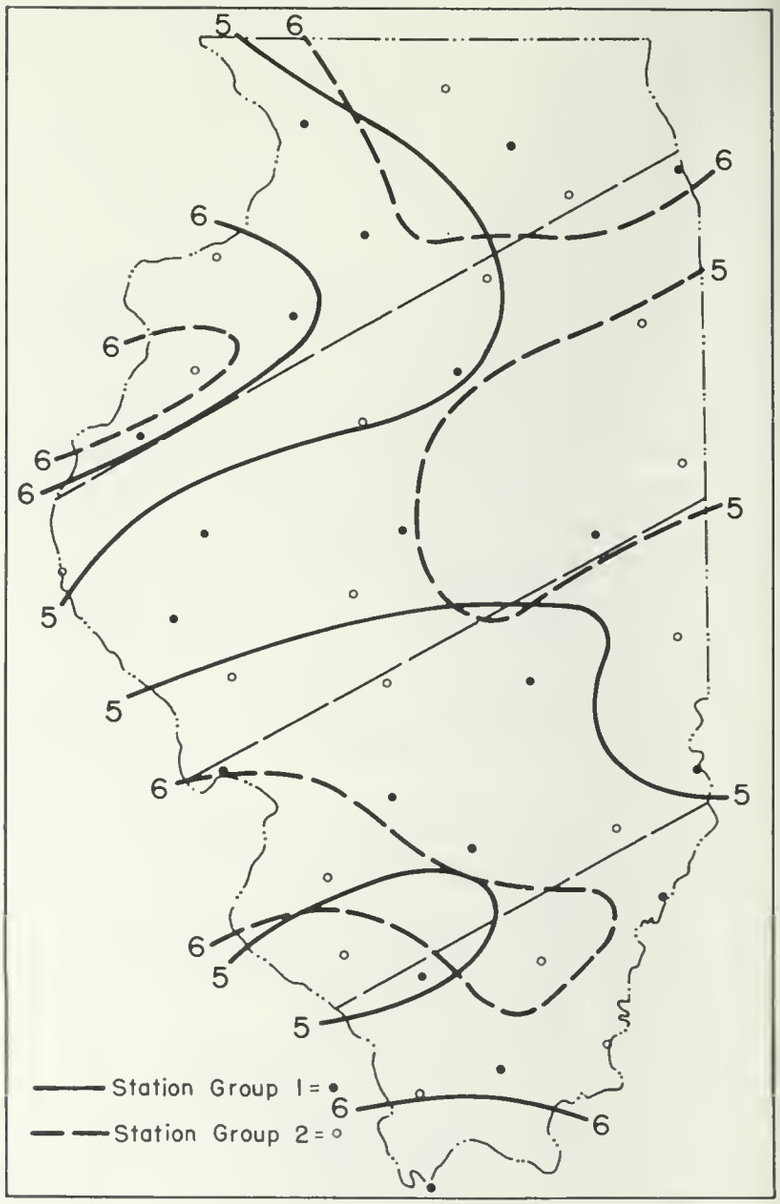

FIGURE 4 COMPARISON OF TWO 40-YEAR ISOHYETAL PATTERNS USING GUMBEL METHOD

variation is illustrated in Figure 4, where station data for the 40-year sampling period have been divided into two groups by the random start sampling technique,[8] and isohyetal patterns drawn for a 50-year recurrence interval of 1-day precipitation with each data group, obtained by fitting the data to the Gumbel distribution.[4] Appreciable differences in the orientation and distribution of isohyets are evident, especially in the northeast and southwest regions of the state.

Another example to emphasize the desirability of area averages is presented in Table 1, where the range of the maximum 40-year values observed among various stations in each section is shown. Obviously, the location of a station frequency curve is considerably affected by the magnitude of these extreme values, and the relatively large differences which may be obtained between adjacent stations is ap-

TABLE 1

RANGE OF ONE-DAY, 40-YEAR MAXIMUM PRECIPITATION

	Section			
	Northwest	North Central	South Central	Southeast
Range (In.)	4.65-10.48	3.71-6.61	4.42-8.24	4.25-6.40
Number of Stations	9	13	10	7

parent. For example, the Gumbel distribution fitted to Rockford and Sycamore data in the Northwest Section (Fig. 1) provides 50-year recurrence values of 6.7 inches and 4.7 inches, respectively. If the Frechet method is used, the values become 8.5 inches and 5.2 inches respectively. Analysis of climatological and topographical factors indicates no cause for such marked differences, which are probably due to logical and normal sampling variations. The topography of Illinois is illustrated in Figure 5.[9]

If one assumes that the average 100-year maximum is greater than the average 40-year maximum at stations, then in a 40-year period such as used in this study, one would expect some of the stations in a representative sample to experience maximum precipitation considerably above their true 40-year average while others should fall appreciably below their true 40-year average. However, if one has chosen a representative sample from an approximately homogeneous area, the average relations developed from the data should provide a reliable estimate of the true frequency relations at points in the selected area. Furthermore, the distribution of the individual points about the mean frequency curve should then provide a measure of the range of values to be expected within the selected area for given recurrence intervals.

Many other examples, similar to that for Rockford and Sycamore, could be presented using data tabulated for this report and data collected by the Illinois State Water Survey on field surveys of severe rainstorms during the past several years. Perhaps, the results of field surveys in recent years have emphasized the desirability of multiple samples and average relations better than any other factors. Some of the implications of these field survey results will be discussed in a later section in conjunction with the selection of the most applicable statistical distribution for use with the Illinois data.

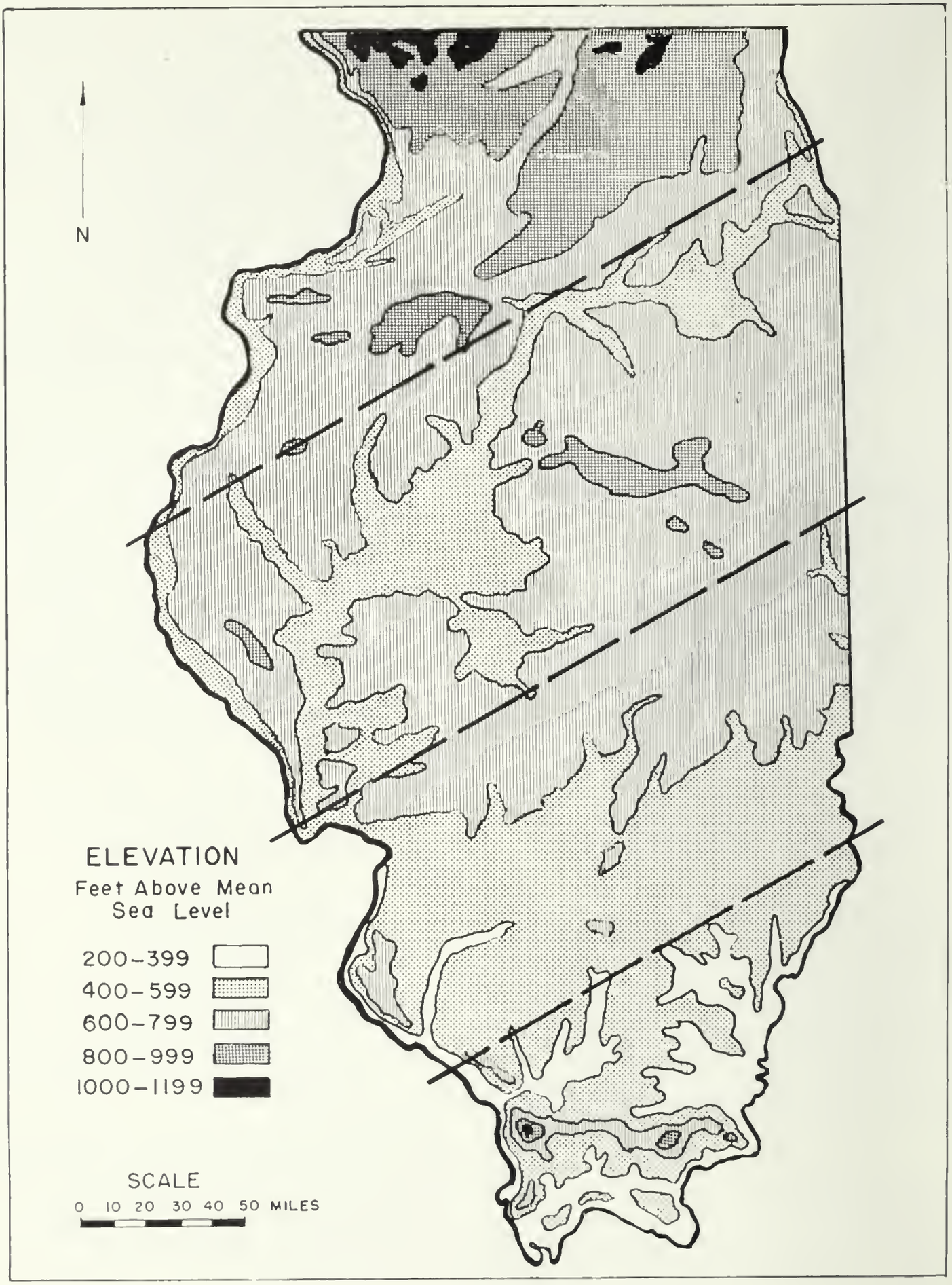

FIGURE 5 ILLINOIS TOPOGRAPHIC MAP

SUMMARY OF FREQUENCY DISTRIBUTION THEORY

Frequency distributions can be classified as either symmetrical or skewed. When a distribution is symmetrical, the mean, mode, and the median coincide. These three parameters are generally separated in the case of skewed or asymmetrical distributions. The amount of separation will depend upon the degree of skewness. Hydrometeorologic variables display various degrees of skewness.

Samples of annual and seasonal maximum rainfall values are expected to exhibit considerable skewness. Skewness is expected even if the maximum values have been drawn from one of the family of normal distributions. This feature of samples of large values may be realized intuitively. When samples are drawn from the upper tail of a unimodal parent population whose upper limit is infinitely large, the probability of getting the larger of the extreme values is less than the chance of getting relatively smaller values. Consequently in repeated sampling, the sample values will tend to be clustered close to the lower end of the large value range. This suggests that large value samples are expected to exhibit various degrees of skewness to the right, i.e., the modal value will be displaced to the left of the median and the median value will be displaced to the left of the mean.

The analysis of extreme value data is concerned with testing the goodness-of-fit of skew distributions and with data transformations which will allow skew data to conform to the characteristics of the normal distribution. In the latter case, goodness-of-fit of the normal distribution to the transformed data would be tested. A brief discussion follows for each distribution that was tested for its ability to describe Illinois maximum annual and seasonal rainfall.

Log-Probability Distribution

Log-Normal. The normal frequency distribution is defined by

$$f(x) = \frac{1}{\sqrt{2\pi}\,\sigma} e^{-\frac{(X-m)^2}{2\sigma^2}} \qquad (1)$$

where X is a variate value; m is the population mean; and σ is the standard deviation of the X-variable. The function represents a two parameter family of symmetrical distributions which are completely characterized by m and σ. Changing m will shift the curve to the right or the left without changing the shape. Values for σ measure the degree of concentration of the data about the mean.

The integral or cumulative distribution of Equation (1) will plot as an S-shaped curve when the value of the variable is plotted against cumulative frequency percentage on rectangular coordinate paper. Generally, the same values are plotted on a special graph paper known as probability paper. The cumulative frequency percentage scale is replaced by a modified scale which has the ability to change the cumulative normal distribution into a straight line. The mean value of the distribution plots at the 50 percent point and the slope of the line is proportional to σ.

The normal distribution curve will not adequately describe maximum annual and seasonal rainfall, since these data are considerably skewed. A transformation of the data is necessary to reduce the skewness factor to a value within the limits of chance variation before the normal distribution function can be applied.

A variable is considered to have a log-normal distribution if the logarithm of the variable can be described by the normal distribution function. The mean and the standard deviation of the logarithm of the variables replace m and σ in plotting the cumulative straight line. The two-parameter log-normal distribution was tested for goodness-of-fit to maximum annual and seasonal rainfall under the assumption that skewness of the logarithms was not statistically significant.

The curve fitting can be done by either the method of moments or by the method of least squares. Curve fitting by the method of moments requires sample estimates $\bar{x}$ and s for m and σ, respectively. These estimates are computed from standard formulas for $\bar{x}$ and s. The line may be constructed on logarithmic probability paper from the location of two points. One point may be located at $X=\bar{x}$ and P= 50 percent, where P is the probability of a value being equal to or less than X. A second point may be located at $X=(\bar{x} - s)$ and P= 15.9 percent. Data points may be plotted around the line for visual inspection of goodness-of-fit. Plotting positions for data points are located at X and $P' = M/N+1$, where M is the ascending rank of an individual observation, X, N is the total number of observations, and P' represents an estimated cumulative percentage.

The P' values must be replaced by corresponding standard normal deviates or standard normal frequency factors, before the method of least squares can be used. Standard normal deviates are generally defined mathematically as

$$y = (X - m) / \sigma \qquad (2)$$

The y-deviates are tabulated in any standard normal integral table. The logarithms of the X-variable are determined, and a plot relating log X to y is assumed to linearize the data. The least squares process is then performed on a log X versus y - coordinate system by fitting

$$\log X = \overline{\log X} + b(y - \bar{y}) \qquad (3)$$

to the data, where $\overline{\log X}$ is the mean of the logarithms, b is the slope of the line, and $\bar{y}$ is the mean of the y-values.

Log-Skew. Chow[6] has proposed a method of logarithmic curve fitting which provides for a variable degree of skewness. The process is essentially a way of adjusting the log-normal probability curve for various skew values.

The adjustment for skewness is based on the log-normal distribution in conjunction with a general formula for hydrologic frequency analysis which Chow expressed as

$$X = \bar{x} + \sigma_x \kappa \qquad (4)$$

where X is an observed value of a variable, $\bar{x}$ is the mean value of the variable, σ_x is the standard deviation of the variable, and κ is defined as a frequency factor which depends upon the law of occurrence of the variable being analyzed. A mathematical definition of κ may be expressed as

$$\kappa = (X - \bar{x}) / \sigma_x \qquad (5)$$

When theoretical expressions for the moments, $\bar{x}$ and σ_x of the log-normal distribution are substituted into Equation (5), the following expression for κ is obtained

$$\kappa = \left[(e^{\sigma_z \kappa_z - \sigma_z^2/2}) - 1 \right] / \sqrt{e^{\sigma_z^2} - 1} \qquad (6)$$

in which

$$\kappa_y = (y - \bar{y}) / \sigma_y \qquad (7)$$

where $z = \log_e X$. Equation (7) is therefore analogous to Equation (5) and defines the frequency factor for $\log_e X$ or z which is assumed to be a normally distributed variable. It is now apparent that Equation (6) expresses the frequency factor, κ, in terms of the frequency factor and standard deviation of a normally distributed variable.

Expressing κ in terms of κ_z and σ_z offers an advantage because values of κ_z are tabulated in the normal probability integral table. Consequently, for any tabled probability level of the normal probability integral a corresponding κ_z or normal deviate can be read.

A value for σ_z is the next requirement for estimating κ from Equation (6). By allowing the data to suggest a value for the coefficient of variation, C_v, a theoretical value of the coefficient of skewness, C_s, can be computed from the relation

$$C_s = 3C_v + C_v^3 \qquad (8)$$

developed by Chow.[6] Using a C_s value from Equation (8), a theoretical value of σ_z can be obtained from a Chow relationship between σ_z and C_s expressed as

$$\sigma_z = \sqrt{\left[\log_e \sqrt[3]{1 + C_s/2 + (C_s\sqrt{4 + C_s})/2} + \sqrt{1 + C_s^3/2 - (C_s\sqrt{4 + C_s^2})/2} - 1 \right]} \qquad (9)$$

Theoretical estimates of the original frequency factor, κ, can now be obtained by substituting the single estimate of σ_z into Equation (6) along with as many κ_z values as desired.

Estimated values of the original X-variable can be computed by substituting the κ estimates in Equation (4) along with estimates of $\bar{x}$ and σ_x which can be computed by using standard formulas. Several values of computed X plotted with corresponding values of P (which are associated with κ) will outline the fitted line on log-normal probability paper. The fitted line will be curved, its curvature depending upon the size of the skewness factor used in computing κ.

The process of fitting log-skew probability curves to data is done theoretically by the method of moments, but from a practical standpoint, the method of least squares may be used. In using least squares analysis, values of κ for each P' have to be computed as described above. Then the least squares fitting process is performed on a log X versus κ

coordinated system with a linear trend line being assumed. The application of the log-skew probability curves was tested to a limited extent for goodness-of-fit as compared with the log-normal and other frequency distribution.

Gumbel Extreme Value

Fisher and Tippett[10] developed the theory for the distribution of extreme values. Gumbel[4] was the first to apply the theory in the field of hydrology. Consequently, this application of the extreme value law is known as the Gumbel distribution.

Extreme value theory applies to either the largest or the smallest values of N independent sets of values, where each set is composed of M independent values and all are drawn from the same population. The initial population from which the N extremes are drawn must follow a simple exponential law so that it is bounded neither on the upper nor lower ends and the frequency of occurrence must approach zero as the variable increases or decreases.

In practice, extreme value theory has been applied when some of the above conditions were only partially met. For example, it has been applied in cases where either upper or lower limits were bounded, but the boundary limits were considerably beyond the range of observation. The theory assumes that both N amd M are large.

Gumbel introduced a reduced variable

$$y = \frac{X - \mu}{\beta} \qquad (10)$$

into the Fisher and Tippett[10] probability function and expressed the probability, P, of an observation being less than X as

$$P = e^{-e^{-\frac{(X-\mu)}{\beta}}} \qquad (11)$$

The distribution of the maximum value of X is the first derivative of (11) and may be expressed as

$$p = \frac{1}{\beta} e^{-\frac{(X-\mu)}{\beta}} e^{-\frac{(X-\mu)}{\beta}} \qquad (12)$$

Equation (12) represents a family of bell-shaped curves which are similar to the normal distribution curves except they are skewed. The mode, μ, is displaced to the left of the mean. The β factor is equal to $0.78\sigma_x$ and is somewhat analogous to the standard deviation of the normal distribution in that it provides a measure of dispersion of the maximum values about their mode. Gumbel postulates an average skewness factor of 1.139 for the distribution.

A substitution may be used to simplify Equation (12), and to reduce the two-parameter family of curves to a standard distribution curve analogous to the standard normal distribution. The integral of the standard distribution has been tabled so that corresponding values of P and y are readily available.

Gumbel used the method of moments to obtain the following asymptotic relations for

$$\mu = \bar{x} - \beta\gamma \qquad (13)$$

and

$$\beta = \frac{\pi}{\sqrt{6}\, s_x} \qquad (14)$$

where $\bar{x}$ and s_x are the sample estimates of their corresponding population parameters (mean and standard deviation) and γ is Eulers' constant which is equal to 0.5772.

Moment estimates of μ and β are obtained from substituting sample values, $\bar{x}$ and s_x, in Equations (13) and (14). Moment estimates are then substituted in Equation (12) to obtain a value of y which is a standard normal deviate having the same probability of being exceeded as any corresponding value of the variable. Probabilities that any y-value and the corresponding value for X will be exceeded may be obtained from a table for the integral of Equation (12). Values of P and X will plot as a straight line on a special probability paper with X on one axis and a special probability scale for P or y on the other axis. This method of fitting the Gumbel curve to data follows the method of fitting by moments.

The mean value for the extreme value distribution does not plot at the 50 percent point as is the case with the log-probability law. Integrating to the mean values would include nearly 60 percent of the distribution, depending on the degree of skewness of the sample.

Various authors of papers on fitting curves to extreme value data have suggested that the fitting may be done by the method of least squares. When the sample size is small, this may be a more efficient fitting process than the method of moments. The least squares process is done on a y versus X diagram, where the y-values replace their corresponding P values. Essentially, the y-scale is used to linearize the data which would plot as a curved trend on other diagrams.

It is interesting at this point to note that Chow[6] has shown that the Gumbel distribution[4] may be considered as a special case of the log-probability law. The correspondence of the two distributions occurs at the point where the log-probability distribution has a skewness value of 1.139. The variable

skewness feature of the log-probability appears to allow for greater flexibility in curve fitting.

Frechet Method

Another form of a two-parameter distribution, which is similar to the Gumbel distribution, was proposed by Frechet and discussed by Gumbel.[7] A logarithmic transformation of the data is necessary before the curve fitting process is done. This is analogous to a transformation from the normal distribution to the log-normal distribution. The cumulative probability may be written

$$P = e^{-e^{-\frac{(\log X - M)}{\alpha}}} \tag{15}$$

where

$$M = (\Sigma \log X)/N \tag{16}$$

and

$$\alpha = \pi/\sqrt{6}\ s_{\log X} \tag{17}$$

where

$$s_{\log X} = \sqrt{\frac{\Sigma(\log X - M)^2}{N-1}} \tag{18}$$

which is the standard deviation for the logarithmic values.

The method of fitting the Frechet distribution to a sample of data may be done by either the method of moments or by the method of least squares. The curve fitting processes are analogous to those described for the Gumbel distribution.

Jenkinson Extreme Value

Jenkinson[5] discusses what he considers a defect in the Gumbel extreme value equation. Instead of the special relation between y and X which Gumbel expressed as $y = (X - \mu)/\beta$, Jenkinson suggests a more general relation between y and X which may be written as

$$y = \frac{x - m}{\sigma_1} = \frac{k! - (\log_e [1/(1-p)])^k}{\pm [(2k)! - (k!)^2]^{1/2}} \tag{19}$$

where X is an individual maxima, m is the mean of the maxima, σ_1 is the standard deviation of the maxima, P is the probability of the nonexceedance, and k is expressed as follows

$$\sigma_1/\sigma_2 = 2^k \tag{20}$$

Jenkinson defines the σ_2 factor in Equation (20) as the standard deviation of the greater members in pairs of annual maxima. An estimate of the σ_2 factor may be computed from the data by

$$s_2 = \frac{\Sigma[(X_M - \bar{x})^2(2M-1)]}{\Sigma(2M-1)} \tag{21}$$

where M is the rank of an observed maxima, X, in ascending order.

The ratio σ_1/σ_2 has the effect of adjusting skewness upward and downward from the average skewness of 1.139 in the Gumbel equation. This feature permits the plotting of concave upward and concave downward curves on Gumbel probability paper in addition to the Gumbel straight line.

Empirical Techniques

Scatter diagrams of the data were plotted on several coordinate systems during the early phases of the analysis. Cube root and square root transformations of maximum rainfall were tested graphically. Semi-logarithmic diagrams were tried with rainfall on the linear scale and recurrence interval on the logarithmic scale. The best linear trend line was obtained when the logarithms of rainfall were related to the logarithms of recurrence intervals. Among these, the log-log plot appeared to describe the data well enough to justify including it in the goodness-of-fit test with the theoretical distributions discussed previously.

INVESTIGATION AND EVALUATION OF FREQUENCY ANALYSIS METHODS

Analysis of Statistical Parameters

As part of the analysis in determining frequency relations for Illinois rainfall, computations were made of means, coefficients of variations, skewness coefficients, and Jenkinson[5] shape factors. This was done for storm periods of 1 to 10 days on an annual and seasonal basis for both station and sectional data for 1916-55.

A comparison between means and coefficients of variation on an annual and seasonal basis in the four sections is shown in Table 2. Reference to the means for the annual data show the expected latitudinal trend except for the North Central Section. The increase of precipitation intensity with decreasing latitude is quite evident in the means for winter, where a very significant increase occurs from the Northwest to the Southeast Sections. During the winter, snowfall is frequent in the Northwest Section, while rain accounts for practically all precipitation in the Southeast Section. A latitudinal trend is apparent throughout the four sections in the spring, but not to the extent of the winter trend. In summer, the departure of the North Central Section from the latitudinal trend becomes apparent and maintains itself through the fall season.

Means of annual maxima for 1-day precipitation at individual stations are shown in Figure 6. Similar analyses were performed for other storm periods and for seasons, but have not been reproduced here since they were used primarily for establishing section boundaries within the state.

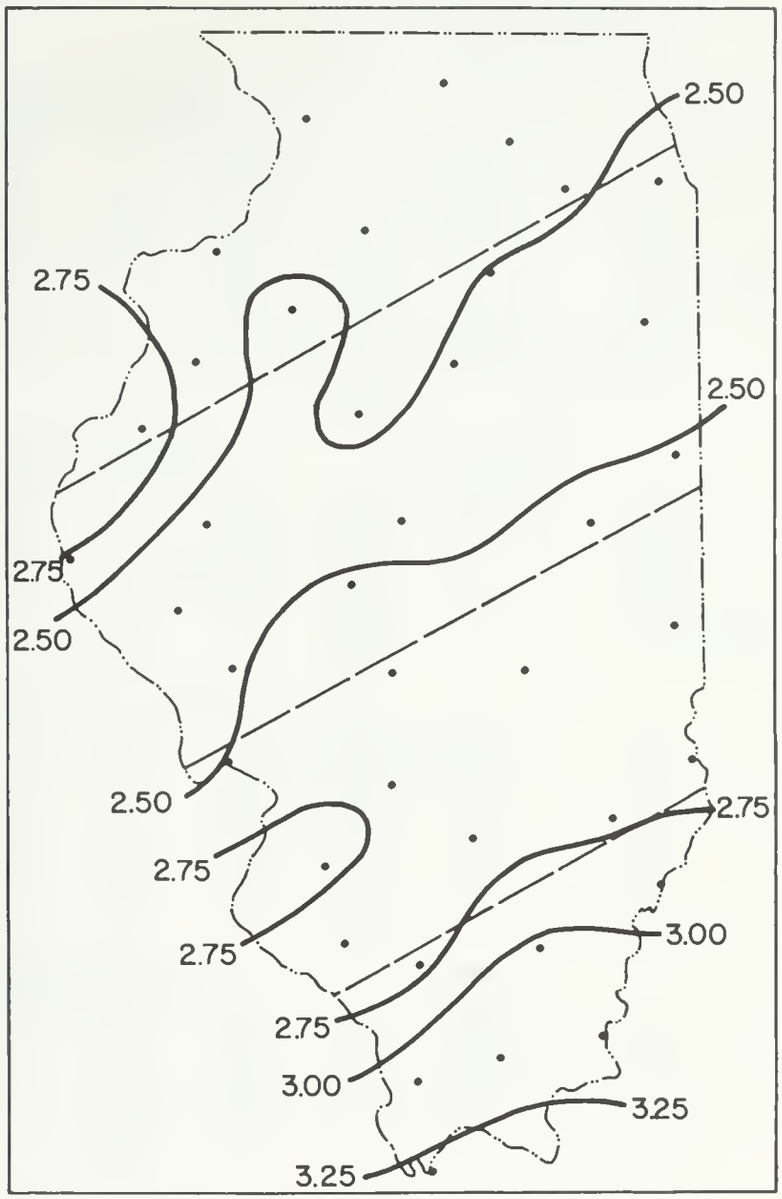

FIGURE 6 MEANS OF ANNUAL MAXIMA FOR 1-DAY PRECIPITATION

TABLE 2

COMPARISONS BETWEEN MEANS AND COEFFICIENTS
OF VARIATIONS, 4 SECTIONS, ANNUAL AND SEASONAL, 1916-55

	Annual		Winter		Spring		Summer		Fall	
	Mean	CV	Mean	CV	Mean	CV	Mean	CV	Mean	CV
1-Day Precipitation										
NW	2.62	0.40	1.02	0.42	1.51	0.39	2.16	0.45	1.86	0.49
NC	2.46	0.33	1.09	0.45	1.60	0.41	1.99	0.44	1.68	0.42
SC	2.68	0.35	1.38	0.43	1.72	0.40	2.05	0.46	1.91	0.45
SE	3.01	0.31	1.89	0.48	2.10	0.41	2.05	0.44	2.01	0.44
Avg.		0.36		0.45		0.40		0.45		0.45
2-Day Precipitation										
NW	3.14	0.37	1.22	0.39	1.91	0.40	2.55	0.45	2.21	0.49
NC	2.95	0.32	1.31	0.47	1.97	0.40	2.35	0.42	2.09	0.44
SC	3.31	0.35	1.69	0.49	2.18	0.41	2.45	0.47	2.35	0.46
SE	3.72	0.29	2.34	0.47	2.68	0.43	2.45	0.45	2.48	0.44
Avg.		0.33		0.46		0.41		0.45		0.46
3-Day Precipitation										
NW	3.40	0.35	1.27	0.40	2.10	0.40	2.79	0.44	2.41	0.46
NC	3.27	0.31	1.39	0.49	2.17	0.39	2.57	0.42	2.33	0.45
SC	3.61	0.34	1.84	0.49	2.40	0.40	2.66	0.48	2.53	0.45
SE	4.07	0.28	2.56	0.47	2.92	0.42	2.61	0.45	2.73	0.44
Avg.		0.32		0.46		0.40		0.45		0.45
5-Day Precipitation										
NW	3.85	0.35	1.45	0.39	2.37	0.37	3.14	0.43	2.69	0.49
NC	3.68	0.30	1.57	0.46	2.48	0.38	2.90	0.41	2.60	0.47
SC	4.04	0.33	2.05	0.47	2.82	0.41	2.99	0.46	2.75	0.44
SE	4.49	0.30	2.93	0.47	3.27	0.41	3.00	0.48	2.88	0.43
Avg.		0.32		0.45		0.39		0.45		0.46
10-Day Precipitation										
NW	4.64	0.35	1.69	0.38	3.00	0.35	3.87	0.38	3.26	0.49
NC	4.49	0.30	1.90	0.42	3.16	0.36	3.57	0.39	3.13	0.47
SC	5.05	0.32	2.53	0.49	3.56	0.43	3.74	0.42	3.34	0.44
SE	5.59	0.29	3.66	0.52	4.05	0.38	3.78	0.45	3.58	0.42
Avg.		0.32		0.45		0.38		0.41		0.46
1 to 10-Day Combined Averages										
NW		0.37		0.40		0.38		0.43		0.48
NC		0.31		0.46		0.39		0.42		0.45
SC		0.34		0.47		0.41		0.44		0.45
SE		0.29		0.48		0.41		0.45		0.43
Over-all		0.33		0.45		0.40		0.44		0.45

Reference to Table 2 shows that coefficients of variation (CV) are relatively constant for precipitation periods of 1 to 10 days. As might be expected, the coefficient is somewhat higher for the seasonal than for the annual data. Among the various seasons, the relative variability appears to be approximately constant except for a slight tendency for a minimum in the spring. Considering both seasonal and annual data, no outstanding trend that one section has greater relative variability than the others is shown. Coefficients of variation of 1-day precipitation for annual maximum precipitation at individual stations are shown in Figure 7.

As an aid in determining the most applicable statistical method of frequency analysis for precipitation periods of 1 to 10 days, skewness coefficients for both individual stations and sectional data were calculated. In addition, the Jenkinson shape factor was calculated. Coefficients of skewness, based upon analysis of station data, are shown in Figure 8 for a 1-day precipitation period. Similar patterns and values were obtained for other storm periods. Results of the sectional analysis are summarized in Table 3, which shows the skewness coefficients (Cs) and shape factors (S_1/S_2) for each section and for annual and seasonal data for 1 to 10 day precipitation periods.

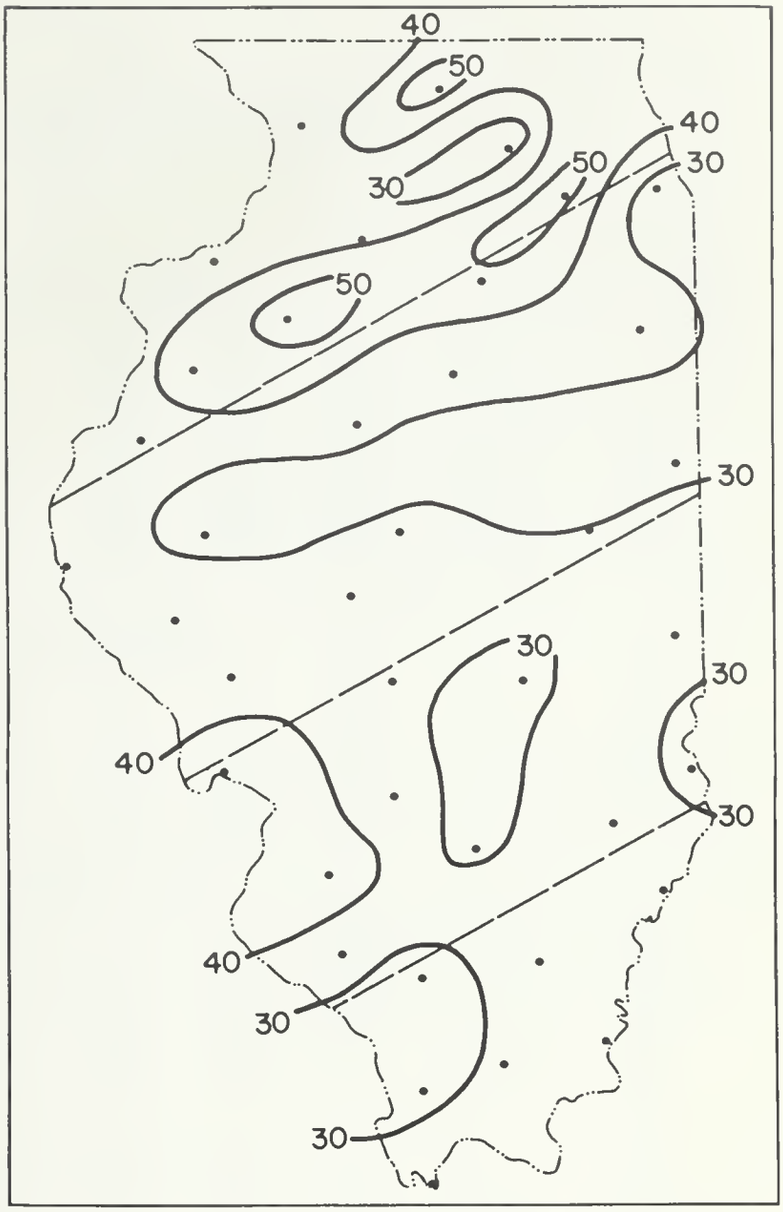

FIGURE 7 COEFFICIENTS OF VARIATION (%) OF ANNUAL MAXIMA FOR 1-DAY PRECIPITATION

TABLE 3

COMPARISONS BETWEEN SKEWNESS AND JENKINSON SHAPE FACTOR, 4 SECTIONS, ANNUAL AND SEASONAL

	Annual		Winter		Spring		Summer		Fall	
	Cs	S_1/S_2	Cs	S_1/S_2	Cs	S_1/S_2	Cs	S_1/S_2	Cs	S_1/S_2
1-Day Precipitation										
NW	1.63	0.84	1.05	0.90	0.71	0.92	1.34	0.86	1.63	0.83
NC	1.14	0.87	1.01	0.88	0.97	0.89	1.08	0.88	1.09	0.88
SC	1.21	0.87	1.02	0.88	0.89	0.91	1.46	0.86	1.14	0.88
SE	0.77	0.91	0.95	0.88	0.71	0.91	1.20	0.87	1.17	0.87
Avg.	1.19	0.87	1.01	0.88	0.81	0.91	1.27	0.87	1.26	0.87
2-Day Precipitation										
NW	1.43	0.86	1.03	0.89	0.74	0.92	1.37	0.86	1.53	0.86
NC	1.06	0.89	1.16	0.87	0.83	0.91	0.93	0.89	1.10	0.88
SC	1.36	0.86	1.33	0.85	0.75	0.91	1.54	0.86	1.20	0.88
SE	0.61	0.92	0.74	0.90	0.83	0.91	0.98	0.88	0.91	0.89
Avg.	1.12	0.88	1.07	0.88	0.79	0.91	1.21	0.87	1.19	0.88
3-Day Precipitation										
NW	1.24	0.87	1.13	0.89	0.69	0.92	1.27	0.86	1.30	0.87
NC	0.91	0.90	1.23	0.87	0.74	0.91	0.86	0.90	1.13	0.88
SC	1.48	0.86	1.35	0.86	0.70	0.92	1.52	0.85	1.24	0.88
SE	0.51	0.93	0.90	0.89	0.66	0.92	1.01	0.89	0.98	0.89
Avg.	1.04	0.89	1.15	0.88	0.70	0.92	1.17	0.88	1.16	0.88
5-Day Precipitation										
NW	1.33	0.86	0.92	0.89	0.52	0.95	1.17	0.88	1.31	0.86
NC	0.93	0.90	0.98	0.89	0.60	0.94	0.81	0.90	1.10	0.88
SC	1.26	0.87	1.41	0.85	0.60	0.93	1.38	0.86	1.21	0.88
SE	0.77	0.91	1.02	0.88	0.71	0.91	1.14	0.87	0.91	0.88
Avg.	1.07	0.89	1.08	0.88	0.61	0.93	1.13	0.88	1.13	0.88
10-Day Precipitation										
NW	1.21	0.84	1.23	0.87	0.18	0.99	0.92	0.90	1.66	0.85
NC	1.06	0.89	0.94	0.90	0.64	0.93	0.71	0.93	1.41	0.86
SC	0.97	0.89	1.16	0.87	0.80	0.90	1.08	0.89	1.24	0.88
SE	0.80	0.90	1.19	0.86	0.53	0.94	1.05	0.88	0.86	0.90
Avg.	1.01	0.88	1.13	0.88	0.54	0.94	0.94	0.90	1.29	0.87
1 to 10-Day Combined Averages										
NW	1.37	0.85	1.07	0.88	0.57	0.94	1.21	0.87	1.49	0.85
NC	1.02	0.87	1.06	0.88	0.76	0.91	0.88	0.90	1.21	0.88
SC	1.26	0.87	1.25	0.86	0.75	0.91	1.40	0.86	1.21	0.88
SE	0.69	0.91	0.96	0.88	0.69	0.92	1.06	0.88	0.97	0.89
Over-all	1.09	0.88	1.09	0.88	0.69	0.92	1.14	0.88	1.22	0.88

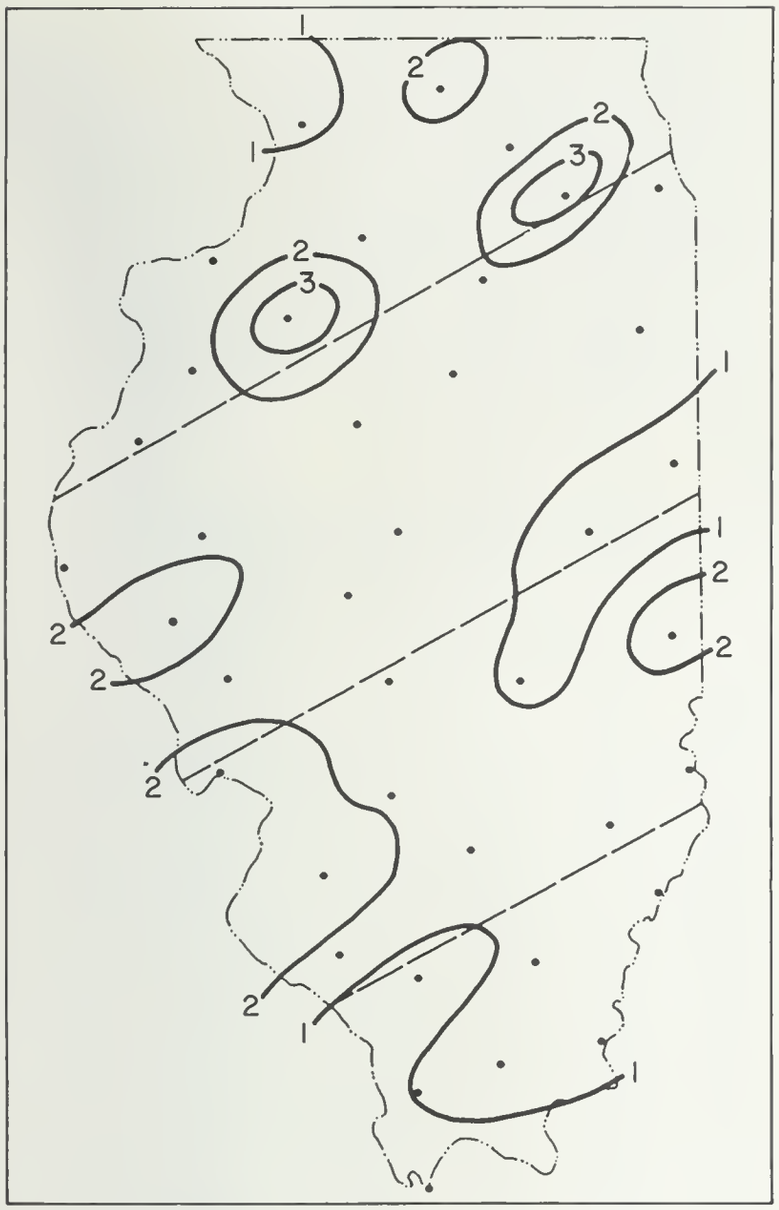

FIGURE 8 SKEWNESS COEFFICIENTS FOR 1-DAY PRECIPITATION

Reference to Figure 8 shows all stations have positive values of skewness with values ranging from less than one to near four. Maximum values were found in the northern portion of the state. Reference to Table 3 shows a tendency for the highest skewness values in the Northwest Section and the lowest in the Southeast for precipitation values of 1 to 10 days on an annual basis. Considerable fluctuation exists within sections on seasonal basis with a tendency for the South Central Section to have highest skewness values in winter, the South Central and North Central Sections in spring, South Central Section in summer, and the Northwest Section in fall. Seasonally, there is a tendency for slightly higher skewness in fall and a slight trend toward a minimum in the spring or summer. Table 3 also shows a trend for the skewness to decrease with length of precipitation period except for the winter season.

Skewness coefficients for the logarithms of the annual maxima are presented in Table 4. These were computed in order to measure the decrease in skewness and the approach to normality when logarithms are used, such as in the log-normal Frechet, and Chow methods tested in this study. Table 4 shows that logarithmic transformations reduced the skewness considerably, but left the majority of the values positively skewed.

TABLE 4

COEFFICIENTS OF SKEWNESS FOR LOGARITHMS OF ANNUAL MAXIMUM PRECIPITATION

	Coefficient for Given Period (Days)				
Section	1	2	3	5	10
NW	0.66	0.55	0.32	0.22	0.52
NC	0.50	0.09	0.14	0.24	0.32
SC	0.40	0.36	0.52	0.37	0.07
SE	0.32	-0.19	-0.03	0.25	0.03

Monthly and Seasonal Distribution of Annual Maxima

An investigation was made of the monthly and seasonal distribution of annual maximum precipitation at each of the 39 stations used in the precipitation frequency study. Station data were then combined to obtain the average distribution for each of the four sections. Results of this investigation are summarized in Table 5. The results were instrumental in making the decision to implement the frequency analysis of annual data with a similar analysis of seasonal relations.

In Table 5 the percent of the total number of occurrences of annual maxima occurring in each month and season are given for storm periods of 2, 5, and 10 days for each section. Thus, in the Northwest Section there are 9 stations or a total of 360 recorded annual maxima for each storm period during the 40 years, 1916-55. In this section, the 2-day annual maximum occurred most frequently in June when 18.6 percent or 67 of the 360 values were recorded. Similarly, 45.3 percent of the 2-day values were recorded during the summer season, June through August.

TABLE 5

MONTHLY AND SEASONAL DISTRIBUTION OF ANNUAL MAXIMUM PRECIPITATION

Percent of Total Cases for Given Storm Period (Days) and Section

	Northwest			North Central			South Central			Southeast		
	2	5	10	2	5	10	2	5	10	2	5	10
Jan.	1.4	0.6	1.1	1.4	1.2	1.4	5.8	7.0	9.3	11.1	13.9	13.2
Feb.	0.6	0.6	0.3	0.8	1.0	0.6	2.0	0.8	1.0	3.6	1.1	1.4
Mar.	8.1	7.2	7.2	7.5	5.2	4.3	5.8	6.5	7.0	12.9	11.8	12.5
Apr.	6.7	5.8	7.0	6.4	10.0	9.3	7.5	11.3	11.3	11.1	11.4	11.1
May	5.8	5 3	8.9	10.0	10.0	13.7	14.0	13.3	16.3	8.2	11.1	10.4
June	18.6	22.0	25.8	15.8	19.1	21.2	11.5	12.0	13.0	7.5	7.5	7.1
July	12.5	13.6	11.4	13.9	12.5	11.2	7.5	8.8	7.0	5.4	5.0	7.5
Aug.	14.2	13.6	11.6	12.9	10.2	10.2	10.3	9.3	12.3	9.3	7.1	8.9
Sept.	16.9	16.9	13.9	14.5	14.1	12.7	12.8	11.0	8.0	9.3	11.1	8.2
Oct.	9.4	9.4	9.4	9.5	8.7	8.7	12.3	11.0	7.8	7.5	7.1	7.9
Nov.	3.9	3.9	3.1	4.4	4.8	4.8	7.5	7.3	6.5	7.9	10.0	7.9
Dec.	1.9	1.1	0.3	2.9	2.7	1.9	3.3	2.0	0.8	6.4	2.9	3.9
Dec.-Feb.	3.9	2.3	1.7	5.0	4.8	3.9	11.0	9.8	11.0	21.1	17.9	18.6
Mar.-May	20.6	18.3	23.1	23.9	25.5	27.3	27.3	31.0	34.5	32.1	34.3	33.9
June-Aug.	45.3	49.2	48.8	42.7	41.9	42.6	29.3	30.0	32.3	22.1	19.6	23.6
Sept.-Nov.	30.2	30.2	26.4	28.4	27.6	26.2	32.5	29.3	22.3	24.6	28.2	23.9

Table 5 indicates that the heaviest storms for periods of 1 to 10 days occur most frequently in June in the Northwest and North Central Sections, with a secondary maximum in September. Average monthly rainfall is also highest during June in these two sections. Summer is the season of most frequent heavy storms in both sections. Moving farther south, the South Central Section shows a maximum storm frequency a month earlier in May, which is the month having the the highest mean rainfall in this part of the state. Similar occurrence frequencies prevail for the spring, summer, and fall seasons in this section for storm periods up to five days, while 10-day maxima are most frequent during spring and summer. In the Southeast Section where the winters are relatively mild the variability among months and seasons is considerably less than in the other sections. Here, the month of maximum frequency is March, closely followed by April and January. March and April are the months of highest average monthly precipitation in this region of Illinois. The season of heaviest storms is spring instead of summer as in the more northerly sections.

Next, the annual maxima among the 39 stations were analyzed for indication of any pronounced trend for increasing or decreasing precipitation intensity during the 40-year period or evidence of any regular cyclic effect in the magnitude of the storms. For this purpose, the upper 10 percent of the annual maxima for each station was used. The station data were then combined to perform a sectional analysis. Results of this analysis are summarized in Table 6, where the distribution of the highest values has been tabulated for storm periods of 2, 5, and 10 days for each of eight 5-year periods comprising the 40 years from 1916 through 1955.

TABLE 6

DISTRIBUTION OF HIGHEST 10 PERCENT OF ANNUAL MAXIMUM PRECIPITATION

Percent of Total Cases for Given Storm Period (Days) and Section

	Northwest			North Central			South Central			Southeast		
Period	2	5	10	2	5	10	2	5	10	2	5	10
1916-20	6	3	3	6	8	4	32	42	25	18	20	4
1921-25	11	8	8	12	13	11	17	10	12	0	4	7
1926-30	22	22	19	19	25	31	3	3	10	7	7	7
1931-35	3	6	0	12	4	4	10	10	3	28	18	14
1936-40	6	11	15	2	4	2	10	5	5	4	11	21
1941-45	11	17	19	21	25	31	10	15	30	14	11	14
1946-50	16	11	17	11	11	9	15	15	15	25	25	33
1951-55	25	22	19	17	10	8	3	0	0	4	4	0

Table 6 does not indicate the presence of any pronounced or regular trend for the precipitation intensity to increase or decrease throughout the 40 years for any of the storm periods. Also no regular cyclic pattern is discernible for these 40 years. Considerable variability of a random or irregular nature does appear, however, and the occurrence frequency appears to correlate well with large-scale climatic events. For example, the frequency of heavy storm periods is relatively low in most cases during the 1930-40 period, which encompassed one of the worst drought periods on record. The 1951-55 frequencies are especially interesting and indicative of the correlation of storm intensity with macro-climatic conditions in the state. During this period, a severe drought was experienced in southern Illinois, while above normal precipitation occurred in the northern part of the state. The correlation is very evident by comparing the frequencies for the South Central and Southeast Sections with those for the Northwest and North Central Sections, in Table 6. Another example of the existing correlation is found in the 1916-20 frequencies. U. S. Weather Bureau records(3) show above normal precipitation for southern Illinois during this period, while the northern portion of the state had considerably below normal precipitation. The correlation between large-scale climatic conditions and the frequency of storm events was also evident in the investigation of the monthly and seasonal frequencies of annual maxima discussed earlier in this section. For example, annual maxima were found to occur most frequently in spring in the Southeast Section. Spring is also the season of heaviest total rainfall in this region.

Frequency Relations for Individual Stations

In the earlier stages of the project, the annual maxima data for individual stations were used to derive frequency relations for each station using the Gumbel, Chow, Jenkinson, log-normal, and Frechet methods. Several other empirical approaches were also tested on a more limited scale by graphical analysis techniques. These included plotting on log-log and semi-logarithmic scales and investigating cube root transformation of the precipitation data.

Results of this phase of the analysis convinced the authors that average sectional relations would be preferable to individual station analysis. Consequently, only a single illustration will be used to indicate the results of this phase. Figure 9 shows the 50-year recurrence values for 1-day annual precipitation, based upon the Gumbel method of analysis. Reference to Figure 9 shows considerable difference between the recurrence values for individual stations in several areas of the state, although there appear to be no climatological or topographical features (Fig. 5) to support these appreciable differences. For example, in the Northwest Section amounts vary from 4.75 to 6.96 inches within a distance of 25 miles from Sycamore to Aurora. In the South Central Section, a value of 6.64 is obtained at Mascoutah while less than 50 miles to the east-northeast a 5.06-inch value is found at Salem. Farther south, values range from 4.66 inches at DuQuoin to 6.25 inches at McLeansboro, about 30 miles to the east. Available evidence indicates that these relatively large dif-

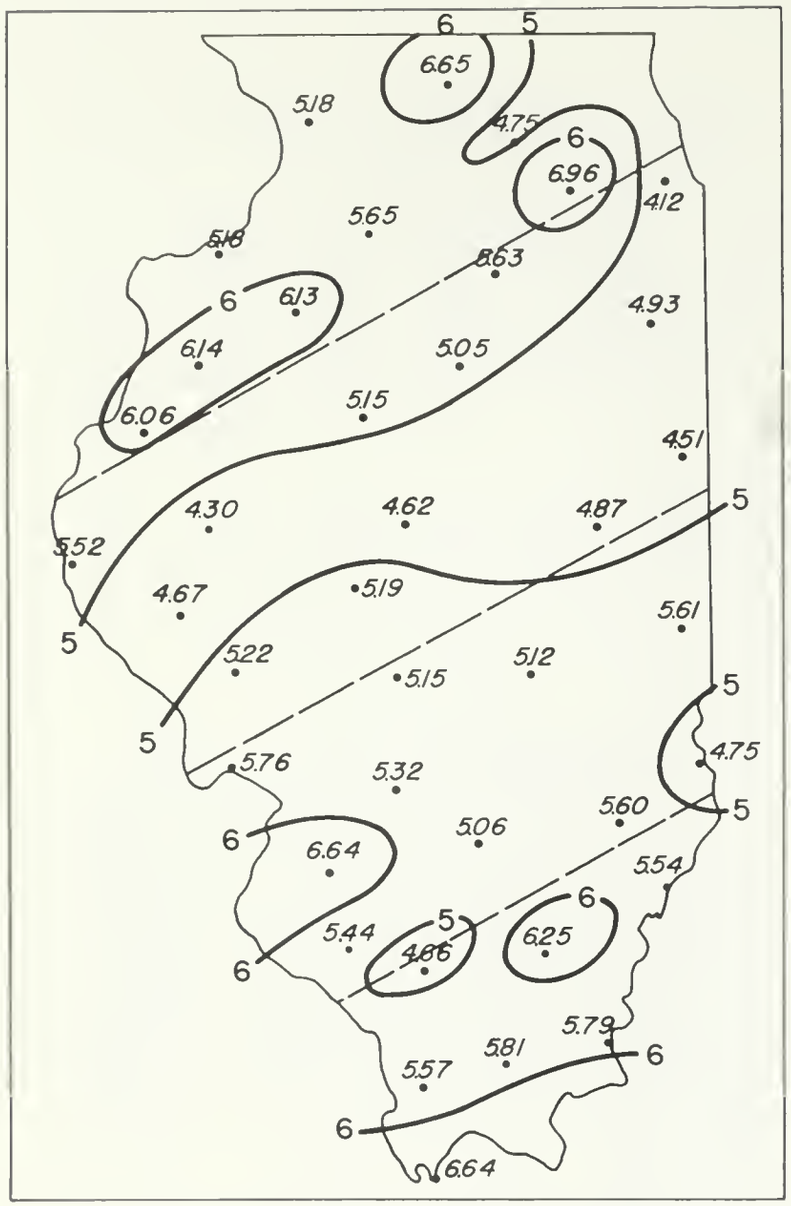

FIGURE 9 50-YEAR RECURRENCE OF 1-DAY PRECIPITATION BASED ON GUMBEL METHOD

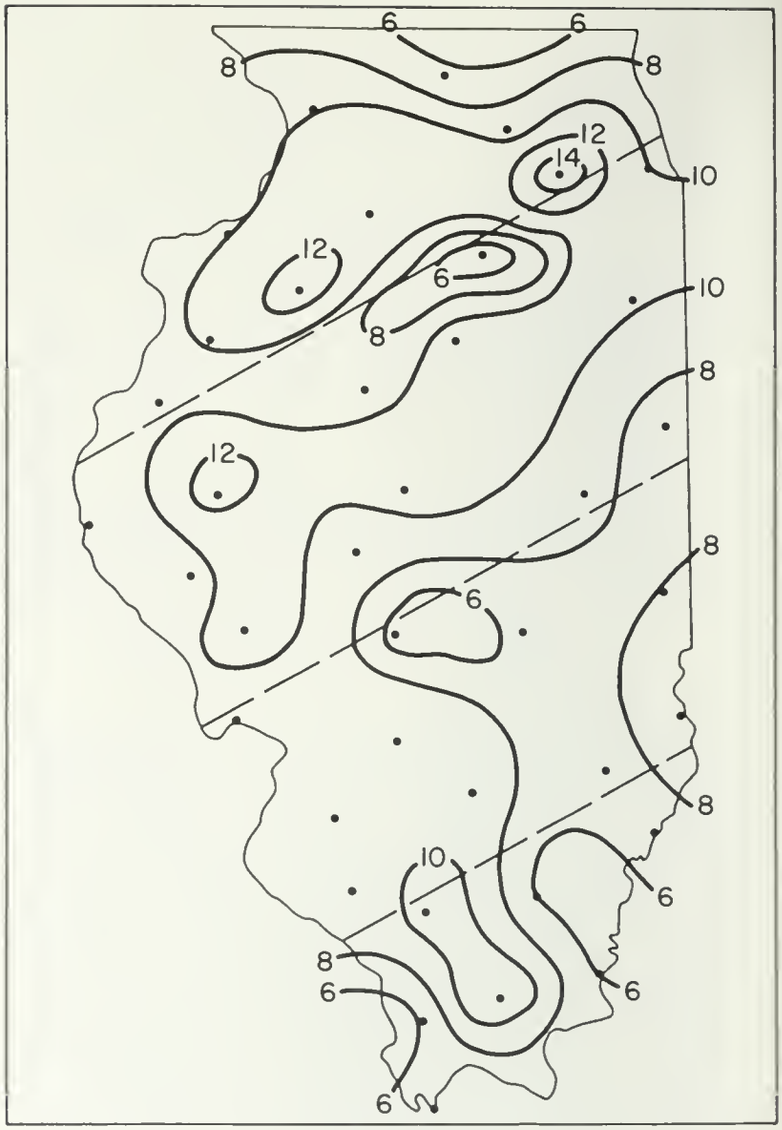

FIGURE 10 MAXIMUM DIFFERENCE (%) BETWEEN 50-YEAR RECURRENCE VALUES OF 1-DAY PRECIPITATION USING FOUR ANALYSIS METHODS

ferences are mainly the result of random spatial fluctuations which may be expected within a given sampling period, rather than any real climatic differences. Therefore, based on available evidence it appears that sectional analysis, in which all data are grouped and a measure of the normal variability range provided, is preferable to an isohyetal map presentation utilizing individual station data.

As a part of the evaluation of various statistical methods applied to frequency analysis, the Gumbel, log-normal, Chow, and Jenkinson methods were applied to individual station data. The raw data were fitted to each distribution by the method of moments. The maximum differences obtained between the four methods for a 50-year return period of 1-day annual precipitation is shown in Figure 10. In this figure, the maximum difference between the methods for each station has been expressed in percent, and calculated by dividing the difference between the highest and lowest return period values by the lowest value. The maximum differences varied from 5 to 15 percent with an average of 9 percent. These maximum differences appear to be quite small, when one considers all the possible sampling variations involved in frequency analysis of station data.

Results of the foregoing analysis indicate little choice between the four statistical methods tested. As discussed earlier in this report, the Chow and Jenkinson methods are more complex statistical techniques. Considering the extra work involved in computations and the small differences obtained in derived frequency relations between these and the

simpler methods involving only the first two moments, use of the Chow and Jenkinson methods did not appear desirable in the present study. Although little difference was found in values predicted by the four methods discussed above, the Frechet method gives values which differ appreciably from these four when the recurrence interval reaches 50 years. These differences will be discussed in detail in a later section.

Comparison Between Two 20-Year Periods

During the early phases of the frequency study, considerable effort was given to finding the most applicable statistical method of frequency analysis. As a part of this phase, a comparison was made between the observed rainfall distribution during 1936-55 and recurrence interval values predicted by various statistical methods utilizing 1916-35 data. The frequency distribution of 1-day, 5-day, and 10-day precipitation amounts were calculated from the 1916-35 data using the Gumbel, log-normal, Chow, Jenkinson, and Frechet methods. In addition, the 1916-35 ranked data were assumed to represent the frequency distribution of future events; that is, the raw data were ranked and the ranked values assumed to represent recurrence interval values. Thus, the highest rainfall value observed in the 1916-35 period was assumed to represent the 20-year recurrence value. The Gumbel, log-normal, Chow, and Jenkinson values were obtained by the method of moments following standard procedures. The Frechet distribution was obtained by graphical analysis.

Data for the second 20-year period, 1936-55, were then ranked. These ranked data were next compared with the appropriate recurrence interval values obtained from the 1916-35 data. The analysis was based on average relationships in each of the four climatic sections.

TABLE 7

AVERAGE 4-SECTION ERRORS (%) IN USING VARIOUS STATISTICAL METHODS APPLIED TO 1916-35 DATA FOR ESTIMATING 1936-55 FREQUENCIES

Method	Error (%) for Given Return Period (Yrs.)				
	20	10	5	2	Over-all
		One-Day Amounts			
Ranked Data	5	7	6	4	6
Frechet	5	6	6	4	5
Gumbel	10	6	5	3	6
Log-Normal	18	13	6	4	10
Chow	17	12	6	3	9
Jenkinson	17	12	9	12	12
		Five-Day Amounts			
Ranked Data	9	4	4	5	5
Frechet	7	4	4	4	5
Gumbel	13	4	3	4	6
Log-Normal	20	10	5	4	10
Chow	21	15	5	5	12
Jenkinson	20	12	9	10	13
		Ten-Day Amounts			
Ranked Data	9	9	6	7	8
Frechet	8	5	7	6	6
Gumbel	13	5	6	6	7
Log-Normal	21	8	6	6	10
Chow	22	10	7	6	11
Jenkinson	19	13	9	8	12

Results of this analysis are summarized in Table 7. In this table, an average error is given in percent for the various statistical methods when applied to the 1916-35 data to estimate 1936-55 frequencies. Reference to Table 7 indicates that the best estimates of the 1936-55 precipitation frequencies were obtained from the ranking of raw data, from the Frechet curve, and from the Gumbel method. Results indicate that little is to be gained by a complex statistical treatment of frequency data. The authors realize, of course, that the results are based only upon two 20-year periods and consideration of additional periods might significantly alter the results.

Comparison of Frequency Relations Based on Periods of 10, 20, and 40 Years

A comparison was made of frequency relations obtained from 10-year and 20-year periods with those obtained from the 40-year period, 1916-55. The 1916-25 and 1916-35 data were used for the two shorter periods. This phase of the study was undertaken to obtain a measure of the effect of sampling period in determining frequency relations. For comparison purposes, frequency relations were determined using the Gumbel method applied to average data for each of the four sections and for 10 individual stations distributed throughout the four sections. The station comparison was accomplished for the 20-year and 40-year periods only. Results of the investigation are presented in Tables 8 to 10.

TABLE 8

COMPARISON BETWEEN SECTION FREQUENCY RELATIONS
DERIVED FROM 20-YEAR AND 40-YEAR DATA USING GUMBEL METHOD

	Difference (%) for Given Recurrence Interval (Yrs.)					
Section	40	20	10	5	2	Absolute Average
			1-Day			
Northwest	+5	+4	+4	+5	+5	5
North Central	0	-1	0	0	+1	0
South Central	-2	-3	-2	-3	-3	3
Southeast	-4	-4	-4	-4	-1	3
			5-Day			
Northwest	+2	+2	+3	+3	+3	3
North Central	-1	-1	0	0	+1	1
South Central	-2	-3	-2	-3	-3	3
Southeast	+1	0	0	-1	-1	1
			10-Day			
Northwest	+13	+12	+12	+10	+8	11
North Central	0	- 1	+ 1	+ 1	+3	2
South Central	- 1	- 2	- 2	- 2	-4	2
Southeast	+ 5	+ 4	+ 3	+ 2	-2	3

TABLE 9

COMPARISON BETWEEN SECTION FREQUENCY RELATIONS
DERIVED FROM 10-YEAR AND 40-YEAR DATA USING GUMBEL METHOD

Section	Difference (%) For Given Recurrence Interval (Yrs.) 40	20	10	5	2	Absolute Average
			1-Day			
Northwest	+4	+4	+6	+7	+11	6
North Central	-3	-2	-1	+1	+ 4	2
South Central	-9	-10	-9	-10	-12	10
Southeast	-4	-4	-2	-1	+ 3	3
			5-Day			
Northwest	+1	+1	+2	+2	+ 5	2
North Central	-4	-3	-2	+2	+ 3	3
South Central	-17	-17	-15	-13	-12	15
Southeast	-4	-3	-3	-3	0	3
			10-Day			
Northwest	+13	+12	+12	+11	+ 8	11
North Central	+ 4	+ 4	+ 4	+ 4	+ 3	4
South Central	-15	-15	-14	-13	-10	13
Southeast	+ 6	+ 6	+ 6	+ 7	+ 7	6

TABLE 10

COMPARISON BETWEEN STATION FREQUENCY RELATIONS
DERIVED FROM 20-YEAR AND 40-YEAR DATA USING GUMBEL METHOD

Station	Difference (%) for Given Recurrence Interval (Yrs.) 40	20	10	5	2	Absolute Average
Moline	+ 7	+ 7	+ 7	+ 8	+ 8	7
Aurora	+34	+32	+27	+22	+ 6	24
Quincy	+13	+13	+12	+11	+ 5	11
Peoria	- 1	0	+ 2	+ 4	+ 9	3
Hoopeston	-14	-13	-11	-10	- 5	11
Urbana	- 6	- 5	- 4	- 4	- 2	4
Grafton	+33	+31	+28	+21	+ 7	24
Windsor	-18	-16	-16	-16	-17	17
Sparta	-14	-13	-11	- 9	- 3	10
New Burnside	- 6	- 6	- 4	- 3	0	4
Mt. Carmel	- 1	- 1	+ 1	+ 3	+ 6	2

In Tables 8 and 9, a comparison of recurrence interval values is given for each of the four sections for 1-day, 5-day, and 10-day precipitation amounts for selected recurrence periods. Percentage differences were found by subtracting the shorter period values from the 40-year values and dividing by the 40-year values. Reference to Table 8 shows only relatively small differences between the 20-year and 40-year data. Only for 10-day amounts in the Northwest Section did the differences exceed five percent.

As would be expected, frequencies derived from the 10-year and 40-year data do not agree as closely as the 20-year and 40-year results. Table 9 shows excellent agreement between the derived frequencies for the North Central and Southeast Sections, and for 1-day and 5-day precipitation periods in the Northwest Section. However, differences are relatively large for the South Central Section. Results of the section analysis indicate that calculated frequency relations become relatively stable when the sampling period is 20 years or greater, and when multiple sampling points are used to define average conditions within an area.

Table 10 shows a percentage comparison between 20-year and 40-year values for 1-day precipitation amounts for 10 selected stations. The results show considerable variability among the stations. For example, only minor differences exist between the recurrence values obtained from 20-year and 40-year data at Mt. Carmel, while at Aurora and Grafton the differences are relatively large. Similar variability among stations was found with other precipitation periods. Comparison of Table 10 with 1-day values of Table 8 further emphasizes the desirability of obtaining section relations based on multiple samples.

Comparison Between Methods of Moments and Least Squares

Perusal of the literature and discussions with prominent statisticians indicate that considerable disagreement exists concerning the best technique of fitting observational data to a selected frequency distribution, such as Gumbel's or Chow's. Some favor the fitting of the data to the chosen frequency distribution by the method of least squares. Others favor use of the method of moments. The question arises as to which is more meaningful, the least-squares fit or one based on the mean and a slope factor determined by a statistic of dispersion about the mean, which is the standard deviation in the case of the normal and log-normal distributions and 0.78 standard deviation in the Gumbel method.

Both the least-squares method and the moments method place the line through the sample mean. However, the method of determining the slope is different. The method of least squares minimizes the sum of squares of deviations about a linear trend line in obtaining the slope. In using the method of moments, the slope is determined by minimizing the sum of squares of deviations about the sample mean value.

Climatological events appear to be governed by natural laws which are very complex and, consequently, it is extremely doubtful that they can be completely defined by a theoretical frequency distribution. This is especially true with precipitation, which is subject to not one, but several physical laws. Statistical analysis methods can provide only a yardstick to help estimate the behavior of natural phenomena. Therefore, it does not seem reasonable to say that one method has a better theoretical basis than the other. It would appear best to test various methods with available data and utilize a least-squares fit with the statistical distribution which appears to be the best yardstick for the existing frequency analysis.

Although the authors favored the least-squares approach to the method of moments, both methods were tested to determine any significant differences between them. Sectional data for the period 1916-1955 were fitted using both techniques with annual and seasonal data. Comparisons were made using the Gumbel, log-normal, Chow and Jenkinson methods. Results of the comparisons are shown for a 1-day precipitation period using annual data for each of the four climatic sections in Table 11. Results of a similar nature were obtained with data for longer precipitation periods and for seasons.

TABLE 11

PERCENT DIFFERENCE BETWEEN METHODS OF LEAST SQUARES AND MOMENTS

1-Day Annual Maximum Precipitation

Recurrence (Yrs.)	Gumbel NW	Gumbel NC	Gumbel SC	Gumbel SE	Log-Normal NW	Log-Normal NC	Log-Normal SC	Log-Normal SE
2	0	0	0	0	0	0	0	0
5	- 1	- 2	- 2	0	+ 1	0	0	+ 2
10	- 2	- 1	- 2	0	- 1	+ 1	0	+ 3
20	- 1	- 1	- 1	+ 1	- 2	0	0	+ 3
40	- 2	- 2	- 2	0	- 2	0	0	+ 4
100	- 2	- 2	- 2	0	- 3	0	0	+ 4
Absolute Avg.	1	1	2	0	2	0	0	3
	Chow				Jenkinson			
2	- 8	- 4	- 5	- 4	+ 3	+ 5	+ 5	+ 5
5	- 9	- 2	- 2	0	+ 5	+ 4	+ 4	+ 3
10	+ 1	+ 2	+ 5	+ 4	+ 8	+ 6	+ 6	+ 5
20	+10	+ 9	+ 9	+11	+12	+ 8	+ 8	+ 6
40	+21	+15	+17	+15	+15	+11	+11	+ 9
100	+34	+26	+21	+23	+17	+12	+13	+ 9
Absolute Avg.	14	10	10	10	10	8	8	6

The percentages in Table 11 were obtained by subtracting moment values from least-squares values and dividing by the least-squares value. This table shows only very small differences between the two methods when applied to Gumbel and log-normal distributions. Differences became appreciable with the Chow and Jenkinson systems, especially at the longer recurrence intervals. Cross-over from negative to positive differences can be noted with the Chow method between the 5-year and 10-year recurrences for all sections and the strictly positive values for Jenkinson throughout. Reference to graphical plots indicated that the Chow and Jenkinson values obtained from the least-squares curves fit the raw data better than those obtained with the moments method, especially above 2-year recurrences which is the region of principal interest to hydrologists.

Comparison of Statistical Distribution Applied to Sectional Annual Maxima Data

In the final evaluation of the several statistical distributions, the method of least squares was applied to sectional data. Use of IBM and digital computor facilities at the University of Illinois greatly facilitated the computations and provided a ready means for the determination of correlation coefficients, standard errors, and coefficients of variation. These parameters were desirable in completing the statistical evaluations prior to final derivation of frequency relations for Illinois rainfall. Least squares regression equations were determined from pooled ranked data for each precipitation period and for annual and seasonal data using the Gumbel, log-normal, and Frechet methods, and an empirical technique in which the logarithms of precipitation are related to the logarithms of the recurrence interval. The latter method was found applicable in fitting partial duration data which will be discussed in a later section and is referred to hereafter as the log-log method. Least-squares regressions were determined for only 1-day precipitation with the Chow (log-skew) and Jenkinson distributions. Previous tests utilizing the method of moments with both station and sectional data indicated that little, if any, improvement could be expected by use of these more complex methods in place of the log-normal and Gumbel methods.

Initially, regression equations were determined for each precipitation period in each section on an annual and seasonal basis. That is,

using annual data for each section there would be regression equations corresponding to precipitation periods of 1, 2, 3, 5, and 10 days for each statistical method tested.

Regression equations were obtained utilizing the method of Chow,[11] whereby the data are fitted to a general linear equation:

$$X = \bar{x} + \sigma_x \kappa$$

Where X is precipitation depth (in.), κ is a frequency factor which varies with the distribution in use (Gumbel, log-normal, etc.), $\bar{x}$ is the mean value and σ_x is the standard deviation of the variable. The factor, κ, is related to the recurrence interval. Convenient tables for conversion with their methods have been prepared by Chow,[6] and Jenkinson,[5] while Weiss[12] has prepared similar tables for Gumbel. The log-normal factors can be obtained from various statistical texts containing normal probability tables.

Inspection of individual equations for 1, 2, 3, 5, and 10 days of annual data for each section indicated that the slopes of the curves within each section were approximately parallel, except for those derived from the Gumbel method which showed a distinct trend for increasing slope with increasing precipitation period. Consequently, single regression equations were determined for the log-normal, Frechet and log-log methods by adding a parameter for precipitation duration. Graphical analysis of data for storm periods of 1 to 10 days indicated that precipitation for a given recurrence interval varied logarithmically with the precipitation period. Therefore, data were combined into one equation of the form:

$$X = ak + b\log T + c$$

Where X is precipitation, κ is a frequency factor as defined by Chow, T is duration of precipitation period in days, and a, b, and c are regression constants.

TABLE 12

CORRELATION COEFFICIENTS FOR REGRESSION EQUATIONS

Period (Days)	Correlation Coefficients for Given Section and Method			
	Northwest	North Central	South Central	Southeast
		Gumbel		
1	0.91	0.95	0.95	0.96
2	0.91	0.96	0.91	0.97
3	0.91	0.96	0.90	0.96
5	0.93	0.96	0.91	0.96
10	0.94	0.95	0.95	0.96
Avg.	0.92	0.96	0.92	0.96
		Log-Normal		
1	0.95	0.96	0.97	0.97
2	0.95	0.96	0.95	0.97
3	0.95	0.97	0.95	0.97
5	0.95	0.96	0.95	0.96
10	0.96	0.96	0.97	0.96
Avg.	0.95	0.96	0.96	0.97
		Frechet		
1	0.96	0.96	0.97	0.96
2	0.96	0.96	0.95	0.96
3	0.95	0.96	0.95	0.95
5	0.95	0.95	0.95	0.96
10	0.96	0.95	0.96	0.95
Avg.	0.96	0.96	0.96	0.96
		Log-Log		
1	0.93	0.93	0.94	0.92
2	0.92	0.93	0.92	0.91
3	0.92	0.92	0.92	0.90
5	0.92	0.91	0.91	0.92
10	0.93	0.92	0.91	0.91
Avg.	0.92	0.92	0.92	0.91
		Chow		
1	0.95	0.96	0.97	0.96
		Jenkinson		
1	0.93	0.96	0.96	0.96

As mentioned previously, several statistical parameters were used to evaluate the statistical distribution methods. Correlation coefficients are presented for individual equations for each period and section using annual data in Table 12. As indicated earlier, calculations for the Chow and Jenkinson distributions were made for only a 1-day precipitation period, because of their complexity which greatly increased the work involved in preparation of the data for analysis with the digital computer. Except for the log-log method, the coefficients in Table 12 show only small differences among the equations derived from the several statistical techniques. Similar results were obtained from comparison of standard errors and coefficients of variation. The analysis to this point established no marked superiority among the several methods in fitting the observational data.

TABLE 13

PERCENT DIFFERENCE BETWEEN RANKED DATA AND REGRESSION CURVE VALUES OBTAINED FROM SEVERAL STATISTICAL METHODS FOR 1-DAY FREQUENCIES

Section	Gumbel	Frechet	Log-Normal	Chow	Jenkinson	Log-Log
			40-Yr. Recurrence			
Northwest	-19	-6	-23	0	-9	+6
North Central	-8	+5	-12	+3	+1	+16
South Central	-16	-3	-19	-4	-8	+7
Southeast	0	+12	-5	-6	+3	+22
Absolute Avg.	11	7	15	3	5	13
			20-Yr. Recurrence			
Northwest	+2	+9	-4	+10	+6	+15
North Central	-7	0	-11	-2	-4	+4
South Central	+1	+8	-3	+7	+4	+12
Southeast	+8	+16	+5	+14	+10	+21
Absolute Avg.	5	8	6	8	6	13
			10-Yr. Recurrence			
Northwest	+2	+1	-2	0	0	+2
North Central	+1	+1	-1	0	-1	+2
South Central	+3	+3	0	+2	+1	+3
Southeast	+5	+6	+3	+4	+3	+9
Absolute Avg.	3	3	2	2	1	4
			5-Yr. Recurrence			
Northwest	+1	-5	-1	-8	-5	-8
North Central	+2	-1	+1	-1	-1	-4
South Central	+1	-2	+1	-2	-2	+8
Southeast	0	-4	-1	-2	-3	-5
Absolute Avg.	1	3	1	3	3	6
			2-Yr. Recurrence			
Northwest	+3	-3	+2	-7	-6	-8
North Central	0	-3	+1	-4	-6	-7
South Central	0	-4	+1	-4	-5	-8
Southeast	+1	-4	0	-4	-5	-6
Absolute Avg.	1	4	1	5	6	7
Over-all Average	4.2	5.0	5.0	4.2	4.2	8.6
Plus	12	9	7	6	7	13
Minus	4	10	11	11	12	7
Zero	4	1	2	3	1	0

In a further attempt to evaluate the statistical methods, a comparison was made between the several methods for the fit of the observational data to the derived curve for recurrence intervals of two years or greater, since this is the region of the curve most useful to the hydrologist. Results of this phase of the analysis are presented in Table 13 for 1-day precipitation. In this table, values for each section and for each statistical method are shown for selected recurrence intervals using annual data. The percent differences were calculated between the ranked observational data and the recurrence curve values. A positive value indicates the curve gave higher values than the ranked data. Similar calculations were made for other precipitation periods but are not shown here since results were very similar.

Except for possible elimination of the log-log method, Table 13 shows little choice between the various statistical methods. Considering all sections and intervals, Frechet shows a more even distribution of positive and negative deviations. Also the data fit is somewhat better than Gumbel, log-normal, and log-log at the extreme upper end of the curve. However, less than one percent difference occurs in the over-all average between the several methods. Consequently, after this analysis it was still not feasible to make a preferential selection of the most applicable statistical distribution.

TABLE 14

COMPARISON OF RECURRENCE INTERVAL VALUES FOR NORTHWEST SECTION

Precipitation (In.) for Given Period (Days)

Recurrence	Gumbel			Log-Normal			Frechet			Chow	Jenkinson
(Years)	1	5	10	1	5	10	1	5	10	1	1
2	2.5	3.7	4.4	2.4	3.7	4.4	2.3	3.5	4.2	2.2	2.2
5	3.5	4.9	5.8	3.4	4.8	5.7	3.2	4.7	5.6	3.1	3.3
10	4.1	5.8	6.8	4.0	5.6	6.6	4.1	5.8	6.8	4.1	4.0
20	4.8	6.6	7.8	4.5	6.3	7.4	5.1	7.0	8.2	5.1	4.9
40	5.4	7.4	8.7	5.1	7.1	8.1	6.3	8.5	9.8	6.7	6.1
100	6.2	8.4	9.9	5.9	8.0	9.0	8.3	10.8	12.4	9.2	7.8

While statistical tests had indicated no significant differences in several of the methods, examination of the calculated precipitation depths showed considerable differences in these values for long recurrence periods. Precipitation values for selected recurrence intervals are shown in Table 14 based on annual data for the Northwest Section. Reference to this table shows 40-year recurrence values ranging from 5.1 to 6.7 inches for 1-day precipitation. Similar differences can be noted for other periods and recurrence intervals in this table. Frechet, Chow, and Jenkinson give larger values for long recurrence intervals. For shorter recurrence intervals, the magnitude of the differences between the methods is not large. One cannot judge whether the the relatively large values predicted for long recurrence intervals are more or less appropriate than the smaller ones.

Although no single method emerged as the best fit to the raw data at this point, the authors favored the Frechet distribution, which gives significantly larger values than the log-normal or Gumbel methods at long recurrence intervals. This fact, of course, is not in itself a good reason for selecting the Frechet method, especially when several methods indicated approximately equal reliability in statistical tests. However, Table 13 shows some evidence in favor of the Frechet method at the upper end of the regression curves, where the differences among distributions become appreciable. The data fit with the Frechet method is illustrated in Figure 11, using 1-day annual maximum precipitation. The open circles represent individual station values and the solid circles are averages.

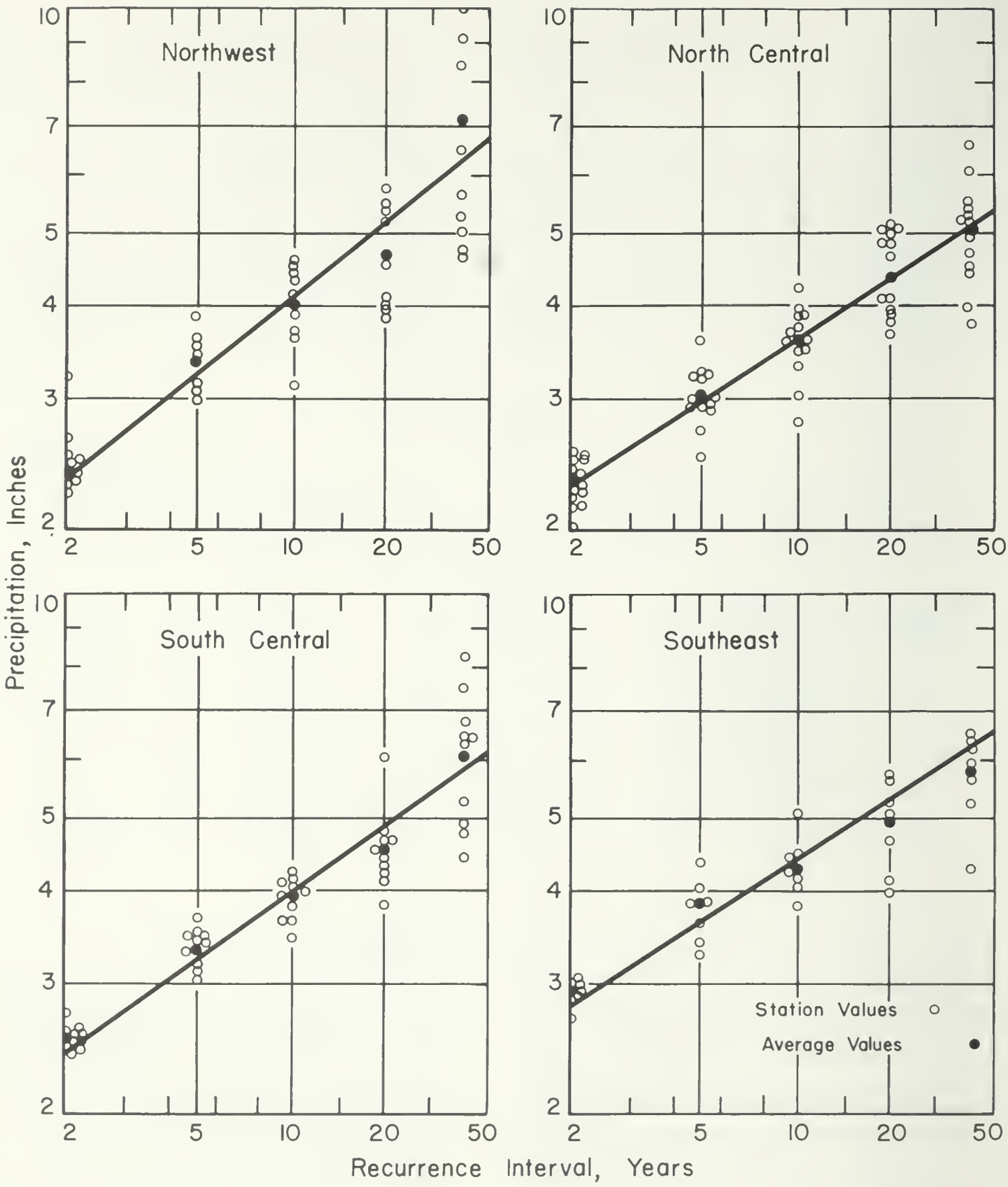

FIGURE 11 ANNUAL MAXIMA ANALYSIS OF 1-DAY PRECIPITATION FREQUENCIES

Frequency Relations Derived from Areal Analyses and Station-Year Method

Since 1948 the Water Survey has been making detailed field surveys of severe rainstorms in Illinois as part of its meteorological program.[13] It was decided to study results from analyses of these storms for indication of the magnitude, extent, and frequency of unusually heavy rainfall amounts within the state. Fortunately, the 10 years for which field survey data are available appear to provide a representative period, with six below normal years and four above normal years of precipitation in Illinois. Isohyetal maps of all severe rainstorms in Illinois for the period 1916-57, obtained from Weather Bureau climatological network data, were also available for assistance in this phase of the frequency investigation.

TABLE 15

AREAL DISTRIBUTION OF HEAVY RAINFALL IN 2-DAY STORMS BY 10-YEAR PERIODS

Area (Sq. Mi.) and Number of Storms with Rainfall Equalling or Exceeding Given Depth (In.)

	6-inch		7-inch		8-inch		9-inch		10-inch	
Period	Area	Number	Area	Number	Area	Number	Area	Number	Area	Number
1948-57	16890	19	9425	13	5035	10	2800	9	1425	7
1938-47	10820	12	5100	4	3170	3	2360	3	1730	1
1928-37	2910	8	270	2	none	0	none	0	none	0
1918-27	12670	10	5990	6	2630	3	260	2	none	0

Basically, an areal analysis is preferable to a point analysis and certainly should provide a more realistic sampling of the magnitude and extent of rainfall at the core of severe storms. The effect of sampling density on the distribution of heavy rainfall amounts is indicated by the comparison of four 10-year periods in Table 15. In this table, the aggregate areas enclosed by isohyets ranging from 6 to 10 inches are shown for each 10-year period. Also, the number of storms in each period having amounts exceeding 6, 7, 8, 9, and 10 inches is indicated. The U. S. Weather Bureau climatological network in Illinois[3] consisted of approximately 90 gages during 1918-27 and was increased to approximately 110 gages by 1937. More rapid increases in gage density followed and by 1942 there were 160 gages in the state. This number was increased to 250 by 1948 and to 280 by 1957. As mentioned previously, detailed field surveys were conducted by the Illinois State Water Survey following the major storms of 1948-57 to provide better definition of the magnitude and distribution of rainfall in the storm centers.

Reference to Table 15 indicates a trend for both enclosed areas and number of storms to increase with increasing sampling density, except for the 1928-37 period which reflects the effects of one of the worst drought periods in the history of the Midwest. One storm in each of the two periods, 1918-27 and 1938-47, produced most of the areal coverage for isohyetal values of 7 to 10 inches; otherwise, the differences between these two periods and 1948-57 would have been much greater. It can be noted that for the 1938-47 period one storm provided more area within the 10-inch isohyet than did seven storms in 1948-57. A 2-day total of 13.77 inches and a 3-day total of 16.57 inches were recorded at Belleville in southwestern Illinois in this August 1946 storm. Field surveys such as conducted since 1948 would undoubtedly have revealed still higher values. These field surveys have shown that storms with amounts exceeding 10-12 inches are not rare occurrences in Illinois.

Consideration was given also to the possible use of the station-year method[11] in the Illinois frequency study. Use of this method requires meteorological homogeneity with respect to area, a satisfactory length of record, and independence of station observations. If all of these conditions are met, supposedly the records from several stations may be combined and treated as a single record equal to the sum of the individual records. The first two of the above requirements are satisfactorily met with the Illinois data, but many of the observations are certainly not meteorologically independent. However, if only the highest 5 to 10 percent of the precipitation values were used for each section in a station-year analysis, the requirement for independence would generally be met. Conceivably, the results of such a station-year analysis could then be used to extend the average sectional curves beyond the 40-year return period for which multiple samples were available to determine the average curves.

Although the above method may appear to be theoretically acceptable, the authors believe the station-year extension technique has a basic weakness which makes use of this method questionable in the present study. The return period values beyond the limits of the average curve would be based upon individual observations at the upper end of the average curve where the sampling is most unreliable. For example, the mean value of 1-day precipitation in the South Central Section is based upon 400 observations since there are 10 stations in this region. The 40-year value on the average frequency curve, however, is defined by only 10 observations, that is, the highest value at each station in the 40-year period. The standard deviation of the 40-year values of 1-day precipitation in the South Central Section is 1.24 inches compared to 0.26 inch at the 10-year recurrence interval and 0.09 inch at the 2-year return period. Furthermore, experience with storm field surveys during the past 10 years has emphasized the inadequacy of rainfall measurements at a few points to sample reliably the unusually severe storms which determine the long recurrence interval values. Results obtained from limited testing of the station-year method will be presented later.

In performing an areal analysis of frequency relations for Illinois, detailed isohyetal maps of 2-day severe rainstorms during 1948-57 were first used. The data for the entire state were combined in this analysis due to the relatively short period of field data available. Analysis was restricted to the area enclosed by isohyets of 6 inches or greater, where the distribution was most accurately defined as a result of the field surveys. Allowance was made for over-lapping isohyets in determining frequency distributions from the areal data.

For comparison purposes, 2-day frequency relations were next determined from data for the 20-year period, 1938-57, after adjusting the 1938-47 data using a gage density factor developed from the 1948-57 data. The gage density factor for each isohyet represented the average ratio of the area enclosed by that isohyet using both climatological network and field survey data to the area enclosed using only climatological data. Most of the severe storms during 1938-47 occurred in the latter part of this 10-year period when the climatological network was not radically different from that during 1948-57. For the purpose of further comparison which might possibly help explain the distribution of heavy rainstorms, the same gage density adjustment was applied to the 1918-37 areal data and the 40-year period, 1918-57, used to develop average 2-day frequency relations for Illinois. Obviously, the gage density was much less, and one would then expect the frequency relations developed from the 1918-57 data to provide smaller values for a given return period, due to the underestimates of high rainfall amounts and coverage.

The station-year method was investigated next and 2-day frequency relations determined for the state using the 1916-55 point rainfall data from the 39 stations available. Both the areal and station-year frequency curves were fitted by graphical analysis. The areal data closely approximated a straight-line fit on a semi-logarithmic scale and the station-year data on a log-log scale. Results from these analyses are presented in Table 16, along with values obtained from the 1916-55 point data using regression analysis to fit the data to the Frechet and Gumbel distributions.

Table 16 shows excellent agreement between the 2-day frequency relations developed from the 1948-57 and 1938-57 areal data and those obtained from the Frechet distribution. As expected, the frequencies obtained from the 1918-57 areal data are considerably below these obtained by the above methods because of inadequate number of raingages during a large part of the period. The station-year values agree closely with the 1918-57 areal values, while the Gumbel distribution provides the lowest values of all the methods, especially at the longer return periods.

TABLE 16

COMPARISON OF 2-DAY AVERAGE FREQUENCIES FOR ILLINOIS DEVELOPED BY POINT AND AREAL ANALYSIS METHODS

Analysis Method	Data Period	Rainfall (In.) Equalled or Exceeded for Given Return Period (Yrs.)			
		25	50	100	200
Areal	1948-57	6.4	7.4	8.5	9.5
Areal	1938-57	6.0	7.1	8.5	10.1
Areal	1918-57	5.5	6.4	7.4	8.6
Station-Year	1916-55	5.7	6.6	7.6	8.8
Frechet	1916-55	6.0	7.1	8.5	9.9
Gumbel	1916-55	5.6	6.3	6.8	7.5

DIFFERENCE (%) FROM FRECHET METHOD

		25	50	100	200
Areal	1948-57	+7	+4	0	-4
Areal	1938-57	0	0	0	+2
Areal	1918-57	-8	-10	-13	-13
Station-Year	1916-55	-5	-7	-11	-11
Gumbel	1916-55	-7	-11	-20	-24

TABLE 17

COMPARISON OF AVERAGE 1-DAY AND 3-DAY FREQUENCIES FOR ILLINOIS DEVELOPED BY POINT AND AREAL ANALYSIS METHODS

Analysis Method	Data Period	Rainfall (In.) Equalled or Exceeded for Given Return Period (Yrs.)			
		25	50	100	200
		1-Day Storm Period			
Areal	1948-57	5.2	6.2	7.2	8.2
Station-Year	1916-55	4.6	5.3	6.2	7.2
Frechet	1916-55	5.0	6.0	7.1	8.3
Gumbel	1916-55	4.7	5.3	5.8	6.4
		3-Day Storm Period			
Areal	1948-57	6.5	7.5	8.6	9.6
Station-Year	1916-55	6.1	7.1	8.2	9.4
Frechet	1916-55	6.6	7.9	9.5	11.0
Gumbel	1916-55	6.0	6.7	7.4	8.2

The analysis procedure described in preceding paragraphs was followed next for 1-day and 3-day storm periods, except that only 1948-57 data were used in the areal analysis since this was the period of most reliable data. No analysis was performed for 5-day and 10-day storm periods due to the lack of field survey data for these periods. Results of the 1-day and 3-day frequency analyses are presented in Table 17. Results are similar to those in Table 16 except that the Frechet values rise considerably above the 1948-57 areal values at the upper end of the curves for 3-day storms, while the station-year and areal values converge in this region of the curves. The Frechet departure at the upper end of the curves is due to the fact that very little difference existed between the 2-day and 3-day amounts in the 1948-57 severe rainstorms, while the 1916-55 point data produces an appreciable difference between these two periods.

The authors realize, of course, that the results presented in Tables 16 and 17 are based upon a limited sample of field survey data, that it is assumed the 1948-57 data represent a near normal period for severe storms, and that it is also assumed a reasonable gage density correction for 1938-47 data can be obtained from 1948-57 data. However, the results do lend support to the selection of the Frechet distribution for analysis of Illinois annual maxima data. The results of the analyses discussed in this section were the basis for the final selection of the Frechet method for this purpose.

The authors do not believe that the Frechet distribution is an unique tool for storm rainfall frequency analysis—it merely seems to provide the most realistic yardstick for analysis within the limits of the data in the present situation. The authors also believe that the Frechet curve rises too rapidly to define accurately the storm rainfall magnitude as return periods reach several hundred years, while the Gumbel and log-normal distributions rise too slowly. This opinion is based upon the analysis of the 1948-57 areal data as well as a consideration of meteorological principles. When an additional 10 years of detailed field survey data are collected in Illinois, a considerably more reliable estimate of storm rainfall distribution in time and space will be possible.

Determination of Seasonal Relations

Seasonal frequency relations were determined by analysis procedures similar to those applied in selection of the Frechet method for the analysis of annual maxima data. Owing to the authors' belief that a statistical distribution can serve only as a yardstick for predicting future events, it was decided to choose a method for seasonal relations that would fit the data best, and not try to make the annual and seasonal data conform to the same distribution.

As a result of the analysis procedures followed it was decided to fit summer data to the Frechet distribution and winter, spring, and fall data to the log-normal distribution using the method of least squares. In all comparisons, there was little choice between the log-normal and Gumbel methods. In general, the statistical tests indicated a slightly better fit of the raw data with the log-normal method, but this difference was of little significance. The statistical tests indicated appreciably better data fit with either the log-normal or Gumbel method than with the Frechet for winter, spring, and fall. Also, the Frechet precipitation values, especially in winter, appeared to increase too rapidly with increasing recurrence intervals to satisfy climatological and meteorological considerations. In the summer when unusually heavy rainfall is most likely to occur over most of the state, the Frechet characteristics appear to fit the frequency distribution better than the other methods, especially for the longer recurrence values. The annual data which are drawn from the highest seasonal value each year also appear to fit the more rapidly rising Frechet curve best at long recurrence intervals, as pointed out earlier. A large portion of the annual values, of course, are drawn from the summer maxima. Statistical comparisons between the three methods are illustrated in Tables 18 and 19, using 1-day winter precipitation. Table 18 shows correlation coefficients for the regression equations for each station, while Table 19 shows a comparison of the raw data fit at selected recurrence intervals.

TABLE 18

CORRELATION COEFFICIENTS FOR 1-DAY WINTER REGRESSION EQUATIONS

	Correlation Coefficients for Given Section and Method			
Method	Northwest	North Central	South Central	Southeast
Gumbel	0.97	0.95	0.93	0.97
Log-Normal	0.98	0.97	0.94	0.98
Frechet	0.96	0.95	0.94	0.96

Determination of Confidence Limits

Methods in current use for computing confidence limits for probability curves have been designed for a single period of record at a single station. For the present analysis, data for the same period of record at several stations were combined to obtain an average probability curve for points in an area.

TABLE 19

PERCENT DIFFERENCE BETWEEN RANKED DATA AND REGRESSION CURVE VALUES FOR 1-DAY WINTER FREQUENCIES

Method	Northwest	North Central	South Central	Southeast	Absolute Average
	Difference (%) for Given Section and Recurrence				
		40-Yr. Recurrence			
Gumbel	-10	-5	-5	-3	6
Log-Normal	-9	-3	-6	+1	5
Frechet	+13	+22	+18	+31	21
		20-Yr. Recurrence			
Gumbel	-4	-6	-6	+1	4
Log-Normal	-5	-6	-6	+1	5
Frechet	+9	+8	+5	+18	10
		10-Yr. Recurrence			
Gumbel	-2	-3	-3	-4	3
Log-Normal	-2	-3	-5	-4	4
Frechet	+1	-1	-2	-1	1
		5-Yr. Recurrence			
Gumbel	+3	+6	+2	+3	4
Log-Normal	+2	+4	0	+5	3
Frechet	-3	-1	-4	-4	3
		2-Yr. Recurrence			
Gumbel	+1	0	+2	+1	1
Log-Normal	0	-1	-1	-2	1
Frechet	-5	-8	-6	-9	7

Combining station data introduces an additional variance factor at each rank which is not accounted for in theoretical confidence limit systems.

Confidence limits which were based on the standard error of estimate from a least-squares fitting process were essentially parallel to the regression curve. The confidence region was too wide in the region of the mean value and too narrow near the data extremes to define adequately the data range. Obviously, the accuracy of predicted extremes is much better in the region of the mean than near ends of the regression curve.

Examination of the data indicated that the standard deviation increased rapidly in magnitude as the rainfall recurrence interval increased. Therefore, it was decided that the range of variability about the regression curve could be defined best by computing standard deviations at numerous recurrence intervals along the regression curve. Logarithms of rainfall were used to achieve approximate normality in determining the standard deviations.

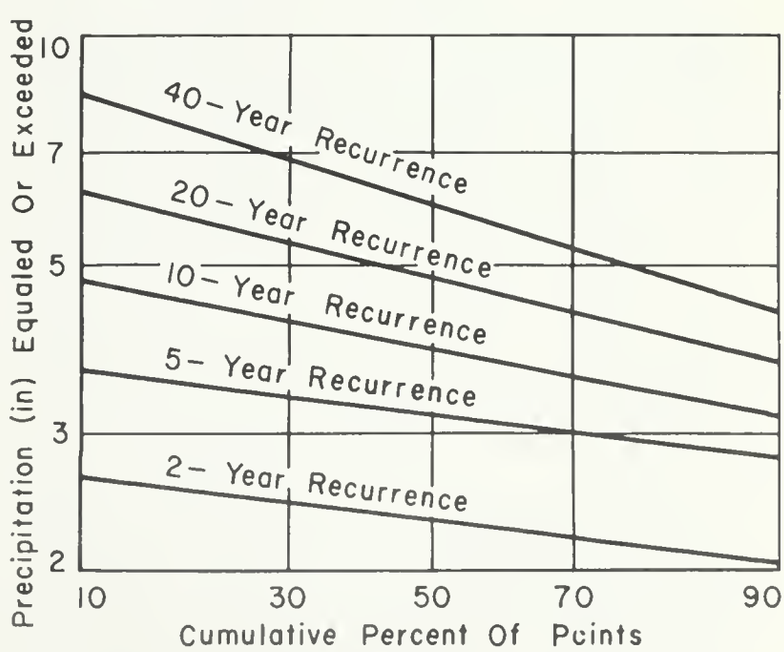

FIGURE 12 PRECIPITATION DISTRIBUTION AT SELECTED RECURRENCE INTERVALS FOR 1-DAY STORMS IN NORTHWEST SECTION

Standard deviation calculations were used in conjunction with regression curve values at various recurrence intervals to construct curves such as those illustrated in Figure 12. In this figure the cumulative percent of points has been related to precipitation at various recurrence intervals for 1-day precipitation in the Northwest Section. Curves such as those shown in Figure 12, in turn, were used to obtain probability curves illustrated in Figure 13. Probability levels of 10 to 90 percent were determined for each section and each precipitation period by the method described.

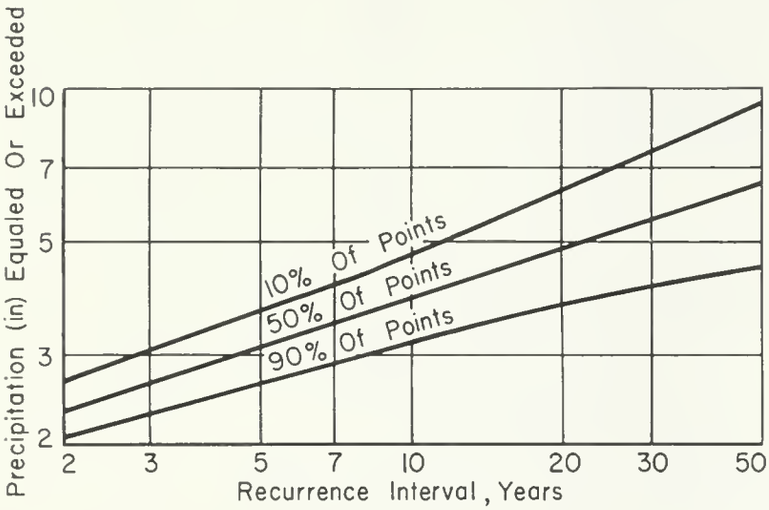

FIGURE 13 PROBABILITY CURVES FOR 1-DAY PRECIPITATION IN NORTHWEST SECTION

Relation Between Annual Maxima and Partial Duration Series

The partial duration series includes all of the high values occurring within a period, whereas the annual maxima includes only the highest value of each particular year within a period. The partial duration series permit use of more than one value in a particular year. Thus, the 50 highest values occurring in a 50-year period will always be included in the partial duration series, but not necessarily among the annual maxima, since the second or third highest values in some years may be greater than the highest value in certain other years. However, processing of partial duration data is very laborious compared to processing of annual maxima data.

Since partial duration values are frequently more useful than those from the annual series, empirical factors are frequently developed to transform annual series values to partial duration values. The U. S. Weather Bureau, for example, has used a selected sample of 50 precipitation stations throughout the United States to develop such transformation factors.[14] As a result of its study, the Weather Bureau found that the ratio of partial duration to annual series values averaged 1.13 for a 2-year return period, 1.04 for a 5-year return period, and 1.01 for a 10-year return period. Beyond a 10-year period, it found the differences between the two series to be insignificant. The Weather Bureau transformation factors apply to periods of 5 minutes to 24 hours.

As part of the Illinois study, data from 10 stations distributed throughout the state were analyzed to obtain average transformation factors for storm periods of one to 10 days. The results of this analysis are summarized in Table 20.

TABLE 20

RATIO OF PARTIAL DURATION TO ANNUAL MAXIMA FREQUENCIES

Precipitation Period (Days)	Ratio for Given Recurrence Interval (Yrs.) 2	5	10
1	1.13	1.05	1.01
2	1.09	1.02	1.01
5	1.09	1.01	1.01
10	1.08	1.01	1.00

The 1-day transformation factors for Illinois in Table 20 are almost identical with those found, in general, for the United States by the Weather Bureau. The ratios decrease slightly with increasing storm period, according to the Illinois study. Comparison of these empirical transformation factors with similar theoretically derived factors[11] shows smaller ratios for the empirical factors. Chow indicates approximate ratios of 1.21, 1.10, and 1.05 for recurrence intervals of 2, 5, and 10 years, respectively, from theoretical considerations. The transformation factors of Table 20 should be used for converting the Illinois maxima values of Tables 23 to 32 to partial duration frequencies. Differences are insignificant for recurrence intervals greater than 10 years.

Comparison Between Calendar-Day and Maximum Period Amounts

To incorporate a large number of stations into the Illinois frequency study, it was necessary to use data from Weather Bureau cooperative stations. Since the majority of these stations have non-recording gages, the frequency analysis has been made on a calendar-day basis. For example, the 1-day frequency relations are based upon 24-hour precipitation totals ending at the time of daily observation at each station. The daily total, of course, does not necessarily include the 24-hour maximum total in a given storm.

Data from six recording gages distributed throughout the state were analyzed for the 10-year period, 1948-57, to determine transformation factors for converting calendar-day to maximum period precipitation. Stations used for this purpose were Rockford, Peoria, Paris, Effingham, Belleville, and Cairo. The transformation factors represent the average ratio of maximum 24-hour, 48-hour, 72-hour, 120-hour, and 240-hour precipitation to 1-day, 2-day, 3-day, 5-day, and 10-day calendar-day amounts, respectively. The ratios are based upon storm periods with precipitation equalling or exceeding 1.5, 2.0, 2.5, 3.0, and 4.0 inches, respectively, for 1, 2, 3, 5, and 10 days. These precipitation values included all those equalling or exceeding the 2-year return period value for the various storm periods, but eliminated the effects of relatively light storms of little interest on calculated transformation factors.

No distinct trend that the transformation factors varied over the state was noted, and as the variability of the factors among stations was sufficiently small, an analysis of a larger sample was not deemed necessary at this time. Furthermore, the factor for converting from 24-hour calendar-day to 24-hour maximum precipitation is the same as that obtained by the U. S. Weather Bureau in other regions of the United States.[14] Results of this analysis are summarized in Table 21. The transformation factors in this table may be used for converting the calendar-day values of Tables 23 to 32 to maximum period precipitation. Except for 1-day storms, however, the differences between calendar-day and maximum period values are hardly significant.

TABLE 21

RATIO OF MAXIMUM PERIOD TO CALENDAR-DAY PRECIPITATION

Storm Period (Days)	Ratio
1	1.13
2	1.02
3	1.01
5	1.01
10	1.01

Duration of Precipitation During Storm Periods of 1 to 10 Days

In problems of utilizing precipitation frequency data for hydrologic design, the question frequently arises regarding the actual duration of rainfall in given storm periods. For example, the 24-hour rainfalls from which a frequency relation is derived may include precipitation periods encompassing a few minutes up to 24 hours. To help explain this problem, the 1948-57 data for six recording stations, discussed in the previous section, were analyzed to determine the actual duration of precipitation during storm periods of various length. Hourly precipitation data from Weather Bureau climatological publications were used in the study.

In the first phase of this analysis, precipitation periods of 1 to 3 days were examined to determine the average number of hours between the beginning and ending of precipitation. Results show that 24-hour rainfall totals on the average occur within a period of 14 hours. That is, the average time from the beginning to the end of the over-all storm is 14 hours, although the actual duration of rainfall may be considerably less than 14 hours, since the storm may be made up of several individual showers within this period. The average number of hours with rain in a storm period will be discussed in the following paragraphs. The average extent of 2-day storms was found to be 30 hours, while 3-day storms averaged 45 hours from beginning to end.

In the second phase of the analysis, the average number of hours having precipitation during storm periods of 1 to 10 days was investigated. Results are summarized in Table 22 which shows that while the average 24-hour storm encompasses a period of 14 hours, precipitation is recorded in only 10 of the 14 hours. Similarly, only 17 of 30 hours have precipitation during 2-day storms and 22 out of 45 hours in 3-day storms. The results of Table 22 may be used with the frequency relations presented in Tables 23 to 32 when a more detailed time distribution is required. Although no distinct trend was noted over the state in the average length of storm period, some evidence was found of a tendency for the number of hours with precipitation in storms to increase southward.

TABLE 22

PRECIPITATION DURATION FOR STORM PERIODS OF 1 TO 10 DAYS

	Storm Period (Days)				
	1	2	3	5	10
Avg. length of precipitation period (hours)	14	30	45	--	--
Avg. number of hours with precipitation	10	17	22	26	32

FREQUENCY RELATIONS FOR STORM PERIODS OF 1 TO 10 DAYS IN ILLINOIS

Frequencies for 1 to 10 Days from Annual and Seasonal Maxima

Final results of the analysis of Illinois frequency relations for storm periods of 1 to 10 days on an annual and seasonal basis are presented in Tables 23 to 32. These results are based upon analysis of the annual maxima series; that is, the annual frequency relations were determined from an analysis of the maximum value recorded in each of the 40 years, 1916-55, while the seasonal frequency relations were based on the maximum value for each particular season in the 40-year period. The frequency relations are also based upon analysis of calendar-day data. Transformation factors for converting from calendar-day to maximum period precipitation and from annual series to partial duration series values are presented in Tables 21 and 20, respectively. Intervals between tabulated values in Tables 23 to 32 are sufficiently small to allow linear interpolation for obtaining recurrence intervals that are not listed. Values given for recurrence intervals of 75 and 100 years in Table 23 should be used with caution, since they were obtained by extrapolating well beyond the limits of the data sample.

TABLE 23

AVERAGE FREQUENCY OF ANNUAL MAXIMUM PRECIPITATION, 1-DAY TO 10-DAY AMOUNTS

Northwest Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	2.3	2.8	3.1	3.5	4.2
3	2.7	3.2	3.6	4.1	4.8
5	3.2	3.8	4.2	4.8	5.7
10	3.9	4.7	5.2	5.9	7.0
15	4.4	5.3	5.9	6.6	7.8
20	4.8	5.7	6.4	7.2	8.6
25	5.1	6.1	6.8	7.7	9.2
30	5.5	6.5	7.2	8.1	9.7
40	5.9	6.9	7.7	8.7	10.4
50	6.4	7.4	8.3	9.3	11.3
75	7.0	8.4	9.3	10.5	12.6
100	7.5	9.0	9.9	11.2	13.2

North Central Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	2.3	2.7	3.0	3.4	4.1
3	2.6	3.1	3.5	3.9	4.7
5	3.0	3.6	4.0	4.5	5.4
10	3.6	4.3	4.8	5.5	6.6
15	4.0	4.8	5.3	6.0	7.3
20	4.3	5.2	5.8	6.6	7.9
25	4.6	5.5	6.1	6.9	8.4
30	4.8	5.7	6.4	7.2	8.7
40	5.1	6.1	6.8	7.8	9.4
50	5.4	6.5	7.2	8.1	9.8
75	6.0	7.2	8.0	9.0	10.9
100	6.4	7.7	8.6	9.8	11.7

TABLE 23 (Cont'd)

South Central Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	2.4	2.9	3.3	3.8	4.5
3	2.7	3.2	3.6	4.1	5.0
5	3.3	3.9	4.4	5.1	6.2
10	4.0	4.8	5.4	6.2	7.6
15	4.5	5.4	6.0	6.8	8.3
20	4.9	5.9	6.6	7.5	9.2
25	5.2	6.2	6.9	7.9	9.7
30	5.4	6.6	7.3	8.4	10.2
40	5.8	7.0	7.8	9.1	10.8
50	6.2	7.4	8.3	9.4	11.6
75	6.9	8.3	9.2	10.5	12.9
100	7.4	8.9	9.9	11.4	13.8

Southeast Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	2.8	3.3	3.7	4.2	5.1
3	3.1	3.7	4.1	4.7	5.6
5	3.7	4.4	4.9	5.6	6.7
10	4.4	5.3	5.9	6.7	8.0
15	4.9	5.9	6.5	7.4	8.9
20	5.3	6.3	7.0	8.0	9.6
25	5.6	6.7	7.5	8.4	10.2
30	5.8	7.0	7.7	8.7	10.6
40	6.2	7.5	8.3	9.3	11.3
50	6.5	7.7	8.5	9.6	11.6
75	7.3	8.7	9.7	11.0	13.2
100	7.8	9.3	10.3	11.8	14.2

The data in Tables 24 to 28 provide a measure of the areal variability of point precipitation within the four sections at selected recurrence intervals for storm periods of 1 to 10 days. Assuming sectional homogeneity, Tables 24 to 28 also provide probability estimates at the 10 to 90 percent levels for any given point. The use of these tables will be illustrated by an example from Table 24. Referring to the 20-year recurrence interval column for the Northwest Section, it is seen that 10 percent of the points (and consequently 10 percent of the sectional area) will have a 1-day precipitation total equalling or exceeding 6.3 inches, while 50 and 90 percent of the points will have amounts equalling or exceeding 4.8 and 3.8 inches, respectively, in an average 20-year period. Since the section is assumed to be meteorologically homogeneous, the percentages in column 1 of Tables 24 to 28 should also represent the probability of any point having precipitation equalling or exceeding the corresponding depth given in the recurrence interval columns. Thus, again referring to the 20-year recurrence column for the Northwest Section in Table 24, it is seen that at any given point in this section, there is a 10 percent probability (one chance in 10) that the 1-day maximum precipitation will equal or exceed 6.3 inches in a 20-year period. Similarly, there is a 50 percent probability that the precipitation at a selected point will equal or exceed 4.8 inches and a 90 percent probability that it will be 3.8 inches or greater in a 20-year period.

TABLE 24

AREAL FREQUENCY DISTRIBUTION OF ANNUAL
MAXIMUM PRECIPITATION, 1-DAY AMOUNTS

Northwest Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	9.4	8.5	7.5	6.3	4.8	3.7	2.7
20	8.5	7.7	7.0	5.9	4.5	3.5	2.6
30	7.7	7.1	6.4	5.5	4.3	3.4	2.5
40	7.1	6.5	6.0	5.1	4.1	3.3	2.4
50	6.4	5.9	5.5	4.8	3.9	3.2	2.3
60	6.0	5.6	5.2	4.6	3.7	3.1	2.3
70	5.6	5.3	4.9	4.4	3.6	3.0	2.2
80	5.1	4.8	4.5	4.0	3.4	2.9	2.2
90	4.6	4.4	4.1	3.8	3.2	2.7	2.1

North Central Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	7.0	6.6	6.0	5.2	4.2	3.4	2.6
20	6.4	6.1	5.5	4.9	4.0	3.3	2.5
30	5.9	5.6	5.1	4.6	3.8	3.2	2.5
40	5.7	5.3	4.9	4.5	3.7	3.1	2.4
50	5.4	5.1	4.8	4.3	3.6	3.0	2.3
60	5.2	4.9	4.6	4.2	3.5	2.9	2.3
70	5.0	4.7	4.5	4.0	3.4	2.9	2.2
80	4.8	4.5	4.3	3.9	3.3	2.8	2.2
90	4.6	4.4	4.1	3.7	3.2	2.7	2.1

TABLE 24 (Cont'd)

South Central Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	7.4	7.0	6.3	5.6	4.5	3.6	2.7
20	7.1	6.7	6.0	5.4	4.3	3.5	2.6
30	6.8	6.4	5.8	5.2	4.2	3.4	2.5
40	6.4	6.0	5.6	5.1	4.1	3.3	2.5
50	6.2	5.8	5.4	4.9	4.0	3.3	2.4
60	5.9	5.6	5.2	4.7	3.8	3.2	2.4
70	5.7	5.4	5.0	4.5	3.7	3.1	2.4
80	5.4	5.1	4.8	4.3	3.6	3.1	2.3
90	5.1	4.8	4.6	4.1	3.5	3.0	2.3

Southeast Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	7.7	7.2	6.7	6.0	4.9	4.1	3.1
20	7.4	6.9	6.4	5.7	4.7	4.0	3.0
30	7.1	6.7	6.2	5.5	4.6	3.9	2.9
40	6.9	6.4	6.0	5.4	4.5	3.8	2.9
50	6.6	6.2	5.8	5.3	4.4	3.7	2.8
60	6.3	5.9	5.5	5.0	4.2	3.6	2.8
70	6.0	5.7	5.3	4.8	4.1	3.5	2.7
80	5.6	5.3	5.0	4.5	3.9	3.4	2.7
90	5.2	4.9	4.7	4.3	3.8	3.3	2.6

TABLE 25

AREAL FREQUENCY DISTRIBUTION OF
2-DAY PRECIPITATION AMOUNTS

Northwest Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	10.0	9.3	8.3	7.1	5.5	4.3	3.0
20	9.4	8.7	7.8	6.7	5.2	4.1	2.9
30	8.7	8.1	7.3	6.4	5.0	4.0	2.9
40	8.0	7.5	6.9	6.0	4.8	3.9	2.9
50	7.4	6.9	6.5	5.7	4.7	3.8	2.8
60	7.0	6.5	6.2	5.5	4.5	3.7	2.8
70	6.6	6.2	5.9	5.3	4.3	3.6	2.8
80	6.2	5.7	5.4	4.9	4.1	3.4	2.7
90	5.7	5.3	5.0	4.6	3.9	3.3	2.7

North Central Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	8.0	7.4	6.8	6.0	5.0	4.1	3.0
20	7.5	7.0	6.4	5.7	4.7	3.9	3.0
30	7.0	6.6	6.1	5.5	4.5	3.8	2.9
40	6.7	6.3	5.9	5.3	4.4	3.7	2.8
50	6.5	6.1	5.7	5.2	4.3	3.6	2.7
60	6.3	5.9	5.5	5.0	4.2	3.5	2.7
70	6.1	5.7	5.4	4.9	4.1	3.4	2.6
80	5.9	5.5	5.2	4.7	4.0	3.4	2.6
90	5.7	5.3	5.1	4.6	3.9	3.3	2.5

TABLE 25 (Cont'd)

South Central Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	11.4	10.4	9.4	8.0	6.1	4.6	3.3
20	9.9	9.2	8.6	7.3	5.6	4.4	3.2
30	9.0	8.3	7.7	6.6	5.2	4.3	3.1
40	8.2	7.6	7.0	6.2	5.0	4.1	3.0
50	7.4	7.0	6.6	5.9	4.8	3.9	2.9
60	7.0	6.6	6.2	5.6	4.6	3.8	2.9
70	6.7	6.3	5.9	5.4	4.5	3.8	2.8
80	6.3	5.9	5.6	5.1	4.3	3.7	2.8
90	6.0	5.6	5.3	4.8	4.2	3.6	2.8

Southeast Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	9.4	8.9	8.3	7.4	6.3	5.2	4.1
20	9.0	8.5	7.9	7.1	6.0	5.0	3.9
30	8.7	8.2	7.6	6.9	5.7	4.8	3.7
40	8.3	7.8	7.3	6.6	5.5	4.6	3.5
50	7.9	7.5	7.0	6.3	5.3	4.4	3.3
60	7.6	7.2	6.7	6.1	5.1	4.3	3.3
70	7.4	7.0	6.5	5.9	5.0	4.2	3.3
80	7.2	6.7	6.2	5.7	4.8	4.1	3.2
90	6.8	6.3	6.0	5.5	4.6	4.0	3.2

TABLE 26

AREAL FREQUENCY OF ANNUAL MAXIMUM
PRECIPITATION, 3-DAY AMOUNTS

Northwest Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	11.4	10.5	9.4	8.0	6.2	4.8	3.3
20	10.6	9.8	8.8	7.6	5.9	4.5	3.3
30	9.8	9.1	8.2	7.2	5.6	4.4	3.2
40	9.0	8.4	7.6	6.8	5.4	4.3	3.2
50	8.3	7.7	7.2	6.4	5.2	4.2	3.1
60	7.7	7.2	6.7	6.0	4.9	4.0	3.1
70	7.2	6.8	6.3	5.7	4.7	3.9	3.0
80	6.7	6.3	5.9	5.3	4.5	3.8	3.0
90	6.2	5.9	5.5	5.0	4.3	3.7	2.9

North Central Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	8.5	8.0	7.4	6.4	5.5	4.5	3.4
20	8.1	7.7	7.1	6.2	5.2	4.3	3.3
30	7.8	7.4	6.8	6.1	5.0	4.2	3.2
40	7.5	7.1	6.6	5.9	4.9	4.1	3.1
50	7.2	6.8	6.4	5.8	4.8	4.0	3.0
60	7.0	6.6	6.1	5.6	4.6	3.9	3.0
70	6.8	6.4	5.9	5.4	4.5	3.8	2.9
80	6.5	6.2	5.7	5.2	4.3	3.7	2.9
90	6.2	5.7	5.4	4.9	4.2	3.6	2.8

TABLE 26 (Cont'd)

South Central Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	12.4	11.4	10.1	8.7	6.6	5.1	3.6
20	11.0	10.2	9.2	7.9	6.2	4.9	3.5
30	9.7	9.0	8.3	7.2	5.8	4.7	3.5
40	8.9	8.4	7.8	6.9	5.6	4.5	3.4
50	8.3	7.8	7.3	6.6	5.4	4.4	3.3
60	7.9	7.4	6.9	6.3	5.2	4.3	3.3
70	7.5	7.0	6.6	6.0	5.0	4.2	3.2
80	7.0	6.4	6.1	5.5	4.7	4.0	3.2
90	6.4	6.0	5.7	5.2	4.5	3.9	3.1

Southeast Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	10.0	9.4	8.8	7.9	6.6	5.5	4.3
20	9.6	9.1	8.5	7.6	6.4	5.3	4.1
30	9.3	8.8	8.2	7.4	6.2	5.2	4.0
40	8.9	8.5	7.9	7.2	6.0	5.0	3.8
50	8.5	8.3	7.7	7.0	5.9	4.9	3.7
60	8.3	8.1	7.5	6.8	5.7	4.8	3.7
70	8.1	7.9	7.3	6.6	5.6	4.7	3.6
80	7.8	7.6	7.1	6.4	5.4	4.6	3.6
90	7.6	7.3	6.9	6.3	5.3	4.5	3.5

TABLE 27

AREAL FREQUENCY OF ANNUAL MAXIMUM PRECIPITATION,
5-DAY AMOUNTS

Northwest Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	12.0	11.0	9.9	8.6	6.8	5.4	3.7
20	11.1	10.3	9.3	8.1	6.5	5.2	3.7
30	10.3	9.6	8.8	7.7	6.3	5.1	3.6
40	9.8	9.1	8.4	7.4	6.1	5.0	3.6
50	9.3	8.7	8.1	7.2	5.9	4.8	3.5
60	8.7	8.1	7.6	6.7	5.6	4.6	3.5
70	8.1	7.6	7.1	6.3	5.3	4.4	3.4
80	7.4	7.0	6.6	5.9	5.0	4.2	3.4
90	6.8	6.4	6.1	5.5	4.8	4.1	3.3

North Central Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	10.1	9.5	8.7	7.6	6.2	5.1	3.8
20	9.6	9.0	8.3	7.3	5.9	4.8	3.7
30	9.1	8.6	7.9	7.0	5.8	4.7	3.6
40	8.6	8.2	7.5	6.8	5.6	4.6	3.5
50	8.1	7.8	7.2	6.6	5.5	4.5	3.4
60	7.8	7.4	6.9	6.3	5.3	4.4	3.4
70	7.5	7.1	6.6	6.0	5.1	4.3	3.3
80	7.1	6.7	6.3	5.7	4.9	4.2	3.3
90	6.6	6.3	5.9	5.4	4.7	4.1	3.2

TABLE 27 (Cont'd)

South Central Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	13.0	12.0	10.8	9.4	7.3	5.6	4.1
20	12.0	11.1	10.1	8.8	7.0	5.4	4.0
30	11.0	10.2	9.4	8.3	6.7	5.3	3.9
40	10.2	9.6	8.9	7.9	6.4	5.2	3.9
50	9.4	9.1	8.4	7.5	6.2	5.1	3.8
60	8.8	8.5	7.9	7.0	5.9	4.9	3.8
70	8.2	7.9	7.4	6.6	5.6	4.7	3.7
80	7.5	7.1	6.7	6.1	5.3	4.5	3.7
90	6.7	6.4	6.0	5.6	5.0	4.4	3.6

Southeast Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	11.4	10.8	10.0	8.9	7.4	6.0	4.7
20	10.9	10.3	9.6	8.6	7.2	5.9	4.6
30	10.4	9.9	9.3	8.4	7.0	5.8	4.5
40	10.0	9.6	9.0	8.2	6.8	5.7	4.4
50	9.6	9.3	8.7	8.0	6.7	5.6	4.3
60	9.2	8.9	8.4	7.7	6.5	5.4	4.3
70	8.8	8.5	8.0	7.4	6.3	5.3	4.2
80	8.5	8.1	7.7	7.0	6.0	5.1	4.2
90	8.1	7.7	7.3	6.7	5.8	5.0	4.1

TABLE 28

AREAL FREQUENCY DISTRIBUTION OF
ANNUAL MAXIMUM PRECIPITATION, 10-DAY AMOUNTS

Northwest Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	13.7	12.8	11.5	10.0	7.9	6.2	4.6
20	13.0	12.1	10.9	9.6	7.6	6.0	4.4
30	12.3	11.5	10.5	9.2	7.4	5.9	4.3
40	11.8	10.9	10.1	8.9	7.2	5.8	4.3
50	11.3	10.4	9.7	8.6	7.0	5.7	4.2
60	10.6	9.8	9.1	8.2	6.7	5.5	4.2
70	9.8	9.2	8.5	7.8	6.4	5.4	4.1
80	9.0	8.5	7.9	7.3	6.1	5.2	4.1
90	8.2	7.8	7.4	6.8	5.8	5.0	4.0

North Central Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	12.3	11.4	10.4	9.0	7.4	6.0	4.5
20	11.6	10.8	9.9	8.7	7.2	5.8	4.4
30	10.9	10.2	9.5	8.4	7.0	5.6	4.3
40	10.3	9.8	9.1	8.1	6.8	5.5	4.2
50	9.8	9.4	8.7	7.9	6.6	5.4	4.1
60	9.4	9.0	8.3	7.5	6.4	5.3	4.1
70	9.0	8.5	7.9	7.1	6.2	5.2	4.0
80	8.3	7.9	7.4	6.7	5.9	5.0	4.0
90	7.5	7.2	6.8	6.3	5.6	4.8	3.9

TABLE 28 (Cont'd)

South Central Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	14.8	13.7	12.4	10.7	8.4	6.7	4.9
20	14.0	13.0	11.8	10.3	8.2	6.5	4.8
30	13.3	12.4	11.3	9.9	8.0	6.4	4.7
40	12.4	11.6	10.7	9.5	7.8	6.3	4.6
50	11.6	10.8	10.2	9.2	7.6	6.2	4.5
60	11.2	10.4	9.8	8.8	7.3	6.0	4.5
70	10.8	10.1	9.4	8.4	7.1	5.8	4.4
80	10.3	9.6	8.9	8.0	6.9	5.6	4.4
90	9.7	9.1	8.5	7.6	6.7	5.5	4.3

Southeast Section

Percent of Area	Precipitation (In.) Equalled or Exceeded for Given Recurrence Interval (Yrs.) 50	40	30	20	10	5	2
10	14.0	12.8	11.9	10.7	8.9	7.5	5.8
20	13.3	12.4	11.5	10.4	8.6	7.3	5.6
30	12.6	12.0	11.2	10.1	8.4	7.1	5.5
40	12.1	11.6	10.9	9.8	8.2	6.9	5.3
50	11.6	11.3	10.6	9.6	8.0	6.7	5.1
60	11.3	11.0	10.3	9.3	7.8	6.5	5.0
70	11.1	10.7	10.0	9.0	7.6	6.4	4.9
80	10.8	10.3	9.7	8.7	7.4	6.3	4.8
90	10.5	9.9	9.3	8.4	7.2	6.1	4.7

TABLE 29

AVERAGE FREQUENCY OF WINTER MAXIMUM PRECIPITATION,
1-DAY TO 10-DAY AMOUNTS

Northwest Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	0.9	1.1	1.2	1.4	1.6
3	1.1	1.3	1.4	1.6	1.9
5	1.3	1.6	1.7	1.9	2.3
10	1.6	1.9	2.1	2.3	2.7
15	1.7	2.1	2.3	2.5	2.9
20	1.8	2.2	2.4	2.7	3.1
25	1.9	2.3	2.5	2.8	3.3
30	2.0	2.4	2.6	2.9	3.4
40	2.1	2.5	2.7	3.1	3.6
50	2.2	2.6	2.8	3.2	3.7

North Central Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.0	1.2	1.3	1.5	1.7
3	1.2	1.4	1.6	1.8	2.1
5	1.5	1.7	1.9	2.2	2.5
10	1.8	2.1	2.3	2.7	3.1
15	2.0	2.3	2.6	2.9	3.4
20	2.1	2.5	2.8	3.1	3.7
25	2.2	2.6	2.9	3.2	3.8
30	2.3	2.7	3.0	3.4	4.0
40	2.5	2.9	3.2	3.6	4.3
50	2.6	3.0	3.3	3.7	4.5

TABLE 29 (Cont'd)

South Central Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.3	1.5	1.7	1.9	2.2
3	1.5	1.8	2.0	2.3	2.7
5	1.9	2.2	2.5	2.8	3.3
10	2.3	2.7	3.0	3.4	4.1
15	2.5	3.0	3.3	3.7	4.5
20	2.7	3.2	3.6	4.0	4.8
25	2.8	3.4	3.8	4.3	5.0
30	2.9	3.5	3.9	4.5	5.2
40	3.1	3.7	4.1	4.7	5.6
50	3.2	3.9	4.3	4.9	5.8

Southeast Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.7	2.1	2.3	2.7	3.2
3	2.1	2.6	2.9	3.4	4.1
5	2.6	3.2	3.5	4.1	4.9
10	3.3	4.0	4.4	5.1	6.2
15	3.6	4.4	4.9	5.6	6.8
20	3.9	4.7	5.3	6.1	7.4
25	4.1	5.0	5.6	6.5	7.9
30	4.3	5.2	5.8	6.8	8.3
40	4.6	5.6	6.2	7.2	8.7
50	4.8	5.9	6.5	7.5	9.2

TABLE 30

AVERAGE FREQUENCY OF SPRING MAXIMUM PRECIPITATION,
1-DAY TO 10-DAY AMOUNTS

Northwest Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.4	1.7	2.0	2.3	2.8
3	1.7	2.1	2.4	2.8	3.4
5	2.0	2.5	2.8	3.2	4.0
10	2.4	3.0	3.3	3.9	4.8
15	2.6	3.2	3.6	4.2	5.2
20	2.8	3.4	3.9	4.5	5.5
25	2.9	3.6	4.0	4.7	5.8
30	3.0	3.7	4.2	4.9	6.0
40	3.2	3.9	4.4	5.1	6.3
50	3.3	4.1	4.6	5.3	6.5

North Central Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.5	1.8	2.1	2.4	2.9
3	1.8	2.2	2.5	2.9	3.5
5	2.1	2.6	2.9	3.4	4.1
10	2.5	3.1	3.5	4.0	5.0
15	2.7	3.4	3.8	4.4	5.4
20	2.9	3.6	4.0	4.7	5.7
25	3.0	3.8	4.2	4.9	6.0
30	3.1	3.9	4.4	5.1	6.2
40	3.3	4.1	4.6	5.3	6.6
50	3.4	4.2	4.8	5.5	6.8

TABLE 30 (Cont'd)

South Central Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.6	2.0	2.2	2.6	3.2
3	1.9	2.4	2.7	3.2	3.9
5	2.3	2.9	3.2	3.8	4.7
10	2.8	3.5	4.0	4.6	5.7
15	3.1	3.8	4.3	5.0	6.2
20	3.3	4.1	4.6	5.4	6.7
25	3.5	4.3	4.9	5.7	7.0
30	3.6	4.5	5.1	5.9	7.3
40	3.9	4.8	5.4	6.3	7.7
50	4.0	5.0	5.6	6.5	8.0

Southeast Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	2.0	2.4	2.7	3.1	3.7
3	2.4	2.9	3.3	3.8	4.5
5	2.8	3.5	3.9	4.5	5.4
10	3.5	4.2	4.7	5.5	6.6
15	3.8	4.6	5.1	6.0	7.2
20	4.1	4.9	5.5	6.4	7.8
25	4.3	5.2	5.8	6.7	8.2
30	4.4	5.4	6.1	7.0	8.5
40	4.7	5.7	6.4	7.4	9.0
50	4.8	6.0	6.7	7.7	9.4

TABLE 31

AVERAGE FREQUENCY OF SUMMER MAXIMUM PRECIPITATION,
1-DAY TO 10-DAY AMOUNTS

Northwest Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.8	2.2	2.4	2.8	3.3
3	2.2	2.7	3.0	3.4	4.0
5	2.7	3.2	3.6	4.1	4.8
10	3.5	4.2	4.6	5.3	6.3
15	4.0	4.9	5.3	6.1	7.1
20	4.5	5.4	6.0	6.8	7.9
25	4.8	5.8	6.4	7.3	8.5
30	5.1	6.2	6.8	7.7	9.0
40	5.7	6.8	7.5	8.6	10.2
50	6.0	7.2	7.9	9.0	10.6

North Central Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.7	2.0	2.2	2.6	3.1
3	2.1	2.5	2.7	3.2	3.7
5	2.5	3.0	3.3	3.8	4.5
10	3.2	3.9	4.3	4.9	5.8
15	3.7	4.4	4.9	5.6	6.6
20	4.1	4.9	5.4	6.2	7.4
25	4.4	5.3	5.9	6.7	7.9
30	4.6	5.7	6.3	7.2	8.4
40	5.0	6.0	6.7	7.7	9.2
50	5.3	6.4	7.1	8.1	9.7

TABLE 31 (Cont'd)

South Central Section

Recurrence Interval (Yrs.)	Depth (In) for Given Period (Days) 1	2	3	5	10
2	1.7	2.1	2.3	2.6	3.1
3	2.1	2.5	2.8	3.2	3.8
5	2.6	3.1	3.4	3.9	4.7
10	3.4	4.1	4.5	5.2	6.2
15	3.9	4.7	5.2	6.0	7.2
20	4.4	5.3	5.9	6.7	8.1
25	4.8	5.7	6.4	7.2	8.6
30	5.1	6.1	6.8	7.6	9.1
40	5.7	6.8	7.6	8.6	10.3
50	6.1	7.2	8.2	9.2	11.0

Southeast Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.7	2.1	2.3	2.6	3.1
3	2.1	2.5	2.8	3.2	3.8
5	2.6	3.1	3.5	3.9	4.7
10	3.5	4.2	4.6	5.3	6.3
15	4.1	4.9	5.4	6.1	7.3
20	4.6	5.5	6.1	6.9	8.3
25	5.0	6.0	6.6	7.4	8.9
30	5.3	6.4	7.0	7.9	9.5
40	5.9	7.0	7.8	8.8	10.7
50	6.2	7.4	8.3	9.3	11.3

TABLE 32

AVERAGE FREQUENCY OF FALL MAXIMUM PRECIPITATION,
1-DAY TO 10-DAY AMOUNTS

Northwest Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.7	2.0	2.2	2.5	2.9
3	2.0	2.4	2.7	3.0	3.6
5	2.5	2.9	3.3	3.7	4.4
10	3.1	3.7	4.0	4.6	5.4
15	3.4	4.0	4.4	5.0	6.0
20	3.7	4.3	4.8	5.4	6.4
25	3.9	4.5	5.0	5.7	6.7
30	4.1	4.7	5.2	5.9	7.0
40	4.3	5.1	5.6	6.3	7.5
50	4.5	5.3	5.8	6.6	7.8

North Central Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.6	1.9	2.1	2.4	2.9
3	1.9	2.3	2.5	2.9	3.5
5	2.3	2.8	3.1	3.5	4.2
10	2.8	3.4	3.8	4.3	5.1
15	3.1	3.7	4.1	4.7	5.6
20	3.3	4.0	4.4	5.1	6.0
25	3.5	4.2	4.6	5.3	6.3
30	3.7	4.4	4.8	5.6	6.6
40	3.9	4.6	5.1	5.9	7.0
50	4.0	4.7	5.3	6.1	7.2

TABLE 32 (Cont'd)

South Central Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.8	2.1	2.3	2.6	3.0
3	2.1	2.5	2.8	3.2	3.7
5	2.6	3.1	3.3	3.8	4.5
10	3.2	3.8	4.2	4.7	5.6
15	3.4	4.2	4.6	5.2	6.1
20	3.8	4.5	4.9	5.6	6.6
25	4.0	4.7	5.3	5.9	7.0
30	4.2	4.9	5.5	6.2	7.3
40	4.4	5.2	5.8	6.5	7.7
50	4.6	5.4	6.0	6.8	8.1

Southeast Section

Recurrence Interval (Yrs.)	Depth (In.) for Given Period (Days) 1	2	3	5	10
2	1.9	2.2	2.5	2.8	3.3
3	2.3	2.7	3.0	3.4	4.0
5	2.8	3.3	3.6	4.1	4.8
10	3.4	4.0	4.4	5.0	5.9
15	3.7	4.4	4.8	5.5	6.4
20	4.0	4.7	5.2	5.9	6.9
25	4.2	4.9	5.5	6.2	7.3
30	4.4	5.1	5.7	6.4	7.6
40	4.6	5.4	6.0	6.8	8.0
50	4.8	5.6	6.3	7.1	8.4

As stated previously, calendar-day values for the annual series in Tables 23 to 32 may be converted to partial duration, maximum period values by the transformation factors given in Tables 20 and 21. The conversion process is illustrated by the following example:

> Assume that it is desired to convert the 1-day, 5-year recurrence interval value for the Northwest Section to its equivalent partial duration, maximum 24-hour value. Reference to Table 23 shows the above recurrence value to be 3.2 inches. First convert from calendar-day to maximum 24-hour amount by multiplying 3.2 inches by 1.13, the conversion factor for 1-day precipitation in Table 21. This results in a value of 3.62 inches. Then multiply 3.62 inches by 1.05, the factor for converting from annual to partial duration series given in Table 20 for a 5-year recurrence interval of 1-day precipitation. This multiplication gives a value of 3.8 inches, the desired partial duration, maximum 24-hour value for the 5-year recurrence interval.

TABLE 33

RATIO OF PRECIPITATION FOR 10, 5, 3, AND 2 DAYS TO 1 DAY

Storm Period (Days)	Ratio for Given Storm Period (Days)				
	Annual	Winter	Spring	Summer	Fall
		Northwest Section			
2	1.19	1.21	1.22	1.21	1.17
3	1.32	1.31	1.40	1.33	1.29
5	1.50	1.50	1.62	1.52	1.47
10	1.80	1.73	1.98	1.79	1.72
		North Central Section			
2	1.20	1.18	1.23	1.20	1.19
3	1.33	1.30	1.40	1.32	1.32
5	1.52	1.47	1.61	1.52	1.53
10	1.82	1.73	1.97	1.82	1.81
		South Central Section			
2	1.21	1.19	1.25	1.21	1.17
3	1.34	1.33	1.39	1.34	1.29
5	1.54	1.49	1.63	1.52	1.46
10	1.86	1.76	2.01	1.82	1.72
		Southeast Section			
2	1.20	1.23	1.22	1.21	1.17
3	1.33	1.35	1.36	1.34	1.31
5	1.52	1.57	1.57	1.51	1.48
10	1.82	1.90	1.90	1.81	1.74

Table 33 is presented to illustrate relations between the several storm periods. Ratios of precipitation for 10, 5, 3, 2 days to 1-day are given on an annual and seasonal basis for each section. The ratios were determined from the frequency relations presented in Tables 23 and 29 to 32. The ratios reach a maximum in spring in all except the Southeast Section, where winter and spring are equal.

The rate of increase of precipitation with increasing recurrence interval is illustrated on an annual and seasonal basis in Table 34, using frequency relations developed for each section. In this table, the ratios of precipitation for 50, 25, 10, and 5 years to 2-year return period precipitation are shown. With respect to the four seasons, the ratios reach a maximum for summer in all sections, indicating the year-to-year variability in the magnitude of storm precipitation is greatest in this season. Combining all sections, spring is the season of lowest ratios or minimum variability with respect to time. All sections have nearly identical ratios in fall, indicating equal variability throughout the state. In extreme southern Illinois, where the heavy storms are

TABLE 34

RATIO OF PRECIPITATION FOR 50, 25, 10, AND 5 YEARS TO 2 YEARS FOR STORM PERIODS OF 1 TO 10 DAYS

Recurrence Interval (Yrs.)	Ratio for Given Recurrence (Yrs.)				
	Annual	Winter	Spring	Summer	Fall
		Northwest Section			
50	2.7	2.3	2.3	3.2	2.6
25	2.2	2.1	2.1	2.6	2.3
10	1.7	1.7	1.7	1.9	1.8
5	1.4	1.4	1.4	1.5	1.5
		North Central Section			
50	2.4	2.5	2.3	3.2	2.5
25	2.0	2.2	2.1	2.6	2.2
10	1.6	1.8	1.7	1.9	1.8
5	1.3	1.5	1.4	1.5	1.5
		South Central Section			
50	2.5	2.6	2.5	3.6	2.5
25	2.1	2.3	2.1	2.8	2.2
10	1.6	1.8	1.8	2.0	1.8
5	1.3	1.5	1.5	1.5	1.5
		Southeast Section			
50	2.3	2.8	2.5	3.6	2.5
25	2.0	2.4	2.2	2.9	2.2
10	1.6	1.9	1.8	2.0	1.8
5	1.3	1.5	1.5	1.5	1.5

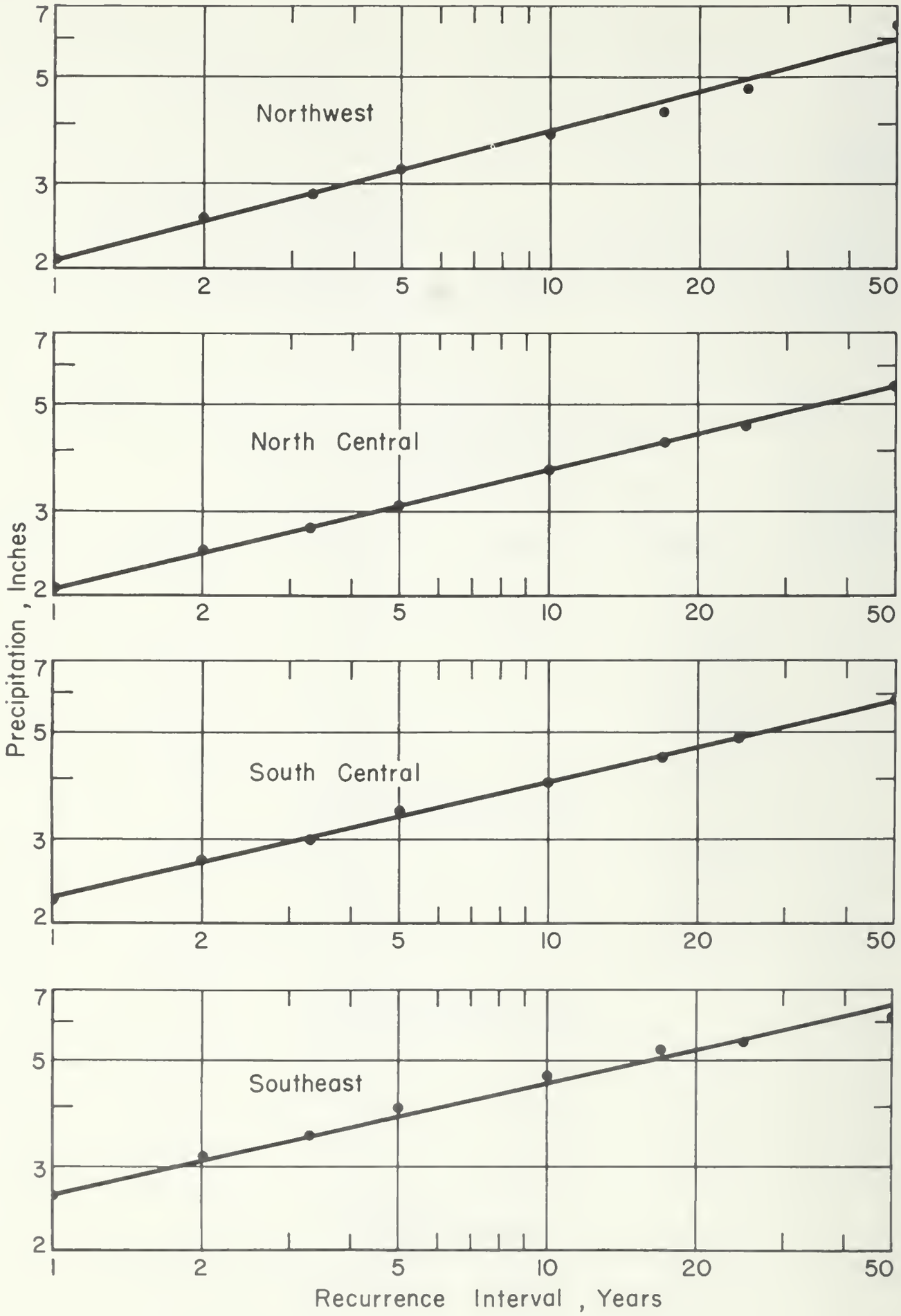

FIGURE 14 PARTIAL DURATION ANALYSIS OF 1-DAY PRECIPITATION FREQUENCIES

distributed more uniformly through the year, the annual ratios are least. Conversely, the highest annual ratios occur in the Northwest Section where most of the heavy storms occur from late spring to early fall.

The magnitude of the ratios for long recurrence intervals is considerably affected by the statistical distribution used in deriving the frequency relations. For example, if the log-normal distribution is used instead of the Frechet method, the 50-year to 2-year annual ratios become 2.0, 1.9, 2.0, and 1.9, respectively, for the Northwest, North Central, South Central, and Southeast Sections. The log-normal 50-year values for 1-day precipitation are 4.9, 4.5, 5.1, and 5.5 inches, respectively, for the above four sections compared to 6.4, 5.4, 6.2, and 6.5 inches obtained from the Frechet analysis presented in Table 23.

Frequencies for 1 Day from Partial Duration Data

A frequency distribution of 1-day precipitation was determined from data for the partial duration series. Analysis of the partial duration series was accomplished with daily precipitation data available on IBM punch cards. Data for 51 stations were available on punch cards at the time of the analysis. These included stations with records spanning a period of 45 to 55 years during the period 1901-1955. The same analysis technique was followed as with the annual maxima data. That is, the state was divided into the same four sections shown in Figure 1 and average frequency relations determined for each section. There were 14, 16, 12, and 9 stations, respectively, available for determining average relations in the Northwest, North Central, South Central, and Southeast Sections.

Frequency curves for each section obtained by applying the partial duration series to annual data are shown in Figure 14. The curves were fitted by graphical analysis. An excellent fit was obtained on a log-log scale, as shown by the position of the plotting points in Figure 14. No further attempt was made to test other curve fitting methods, considering the excellent fit obtained by graphical analysis and the experience gained in the detailed analysis of annual maxima data.

Excellent agreement was obtained between the relations developed from the partial duration series and those determined from annual maxima data for a somewhat shorter and different period of time. Comparison of results from the two methods is presented in Table 35 for selected recurrence intervals. Despite the use of non-equivalent observational periods, the empirical relation between partial duration and annual series frequencies presented in Table 20 is indicated satisfactorily except for the North Central Section.

When the annual series values for periods of 1 to 10 days in Table 23 are converted to partial duration series values by applying the appropriate transformation factors from Table 20, an excellent straight-line fit is also obtained on a log-log plotting scale. However, seasonal data do not fit the log-log relation as satisfactorily when converted to partial duration series values.

The results of the foregoing investigation suggest that simple graphical analysis methods are generally satisfactory for the analysis of frequency data.

Relation Between Annual Precipitation and Storm Frequencies

Investigation was made of the relation between annual precipitation and precipitation frequencies for storm periods of 1 to 10 days. This investigation was made to evaluate further the relations between small-scale and large-scale climatic events, which were indicated in a previous section dealing with the monthly and seasonal distribution of annual maxima.

Annual mean precipitation data for the 50-year period, 1906-55, were used in the analysis. The data for this period were available from an unpublished Illinois State Water Survey study, and the period is of sufficient length to provide a reliable estimate of the population means. The annual precipitation means are presented in Figure 15. The storm period frequencies derived for annual maxima applying the Frechet method (Table 23) were related to the annual means.

First, a station comparison was made between annual mean precipitation and the means of the annual maxima for storm periods of 1 to 10 days. The storm means were expressed as a percentage of the annual means in this phase of the study. Results of this analysis indicated a close association in the four climatic sections. For 1-day precipitation in the Northwest Section, percentages ranged from 7.3 to 8.1 with an average of 7.7. Similarly, the North Central Section had stations ranging from 6.5 to 7.6 with an average of 6.9. In the South Central Section the average was 6.8 with a range of 6.4 to 7.2, while the Southeast Section had an average of 6.9 with a range from 6.4 to 7.7. Combining all four sections, correlation coefficients of 0.74, 0.83, and 0.92 were obtained between mean annual and 1-day, 5-day, and 10-day mean precipitation, respectively.

TABLE 35

COMPARISON OF PARTIAL DURATION AND ANNUAL MAXIMA ANALYSES FOR 1-DAY PRECIPITATION

Recurrence Interval (Yrs.)	Precipitation (In.) for Given Method: Partial Duration	Annual Maxima	Difference (In.)
	Northwest Section		
40	5.7	5.9	0.2
20	4.7	4.8	0.1
10	3.9	3.9	0.0
5	3.2	3.2	0.0
2	2.5	2.3	0.2
	North Central Section		
40	5.2	5.1	0.1
20	4.3	4.3	0.0
10	3.6	3.6	0.0
5	3.1	3.0	0.1
2	2.1	2.2	0.1
	South Central Section		
40	5.5	5.8	0.3
20	4.7	4.9	0.2
10	3.9	4.0	0.1
5	3.3	3.3	0.0
2	2.7	2.4	0.3
	Southeast Section		
40	6.2	6.2	0.0
20	5.3	5.3	0.0
10	4.5	4.4	0.1
5	3.8	3.7	0.1
2	3.1	2.8	0.3

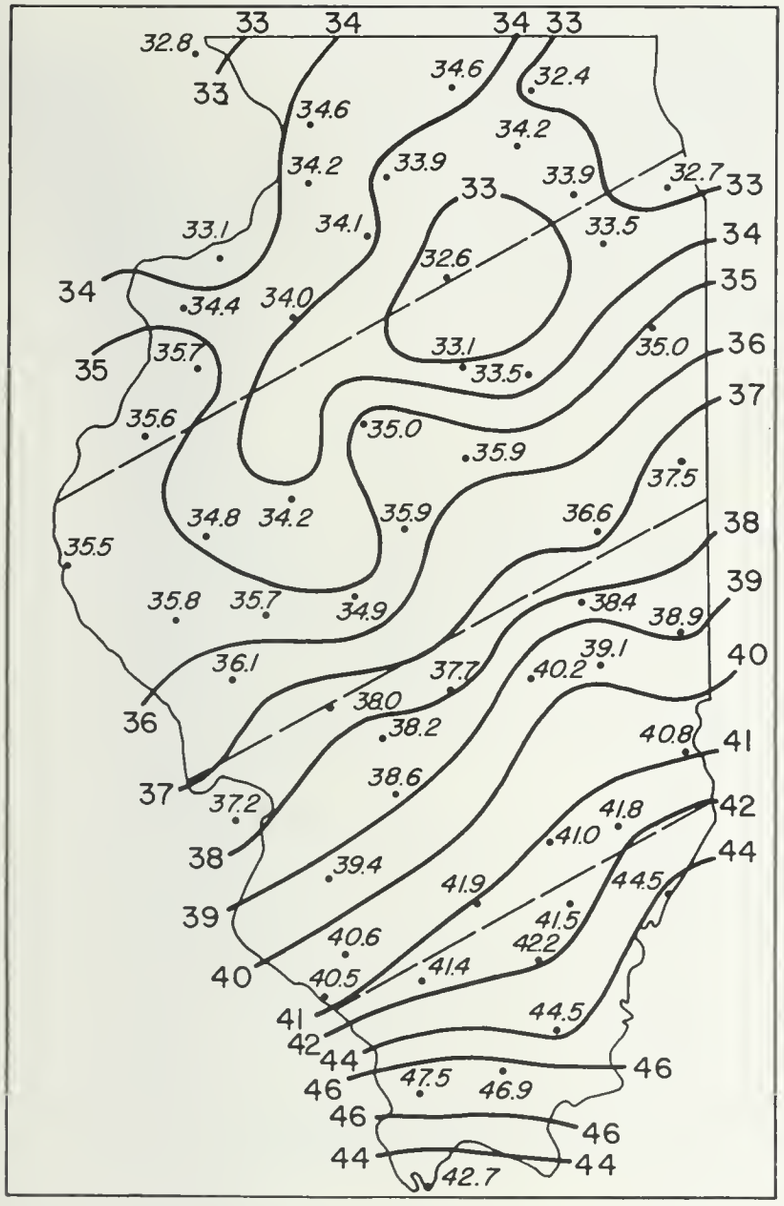

FIGURE 15 MEAN ANNUAL PRECIPITATION, 1906-55

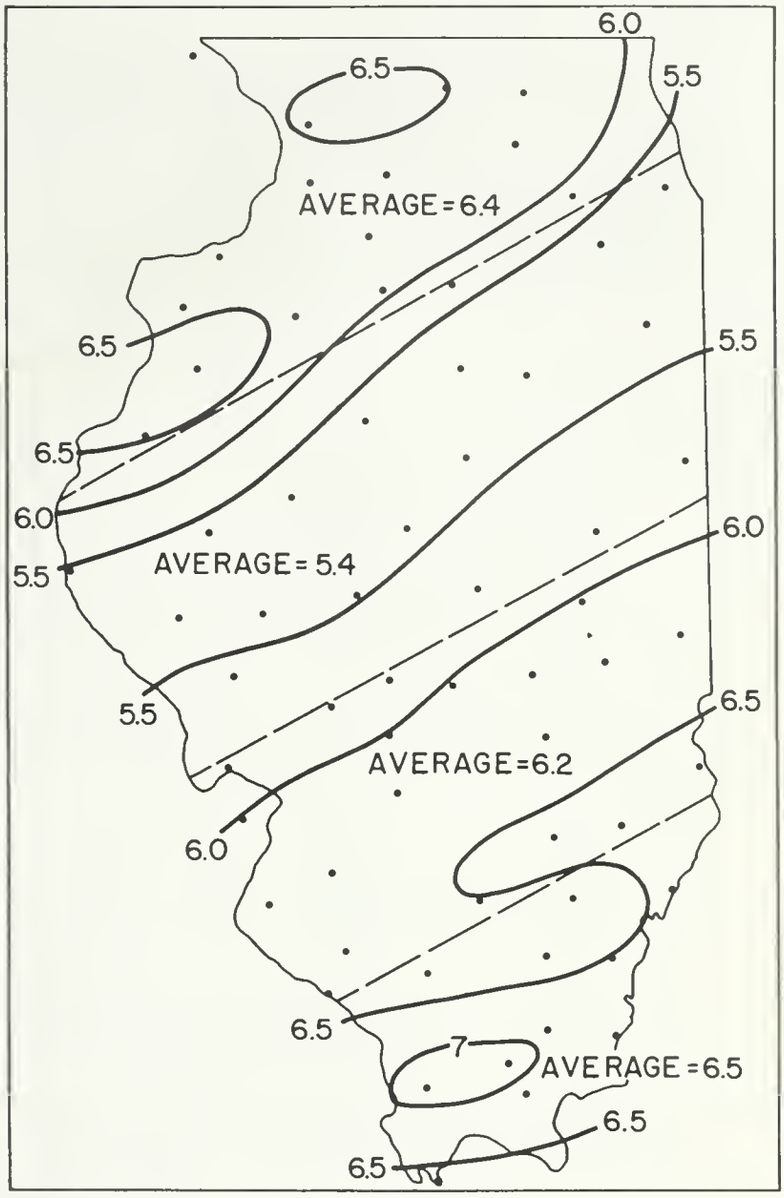

FIGURE 16 50-YEAR RECURRENCE OF 1-DAY PRECIPITATION BASED ON RELATION BETWEEN ANNUAL AND STORM PRECIPITATION

Next, the relation between the average annual precipitation and storm precipitation for various recurrence intervals and storm periods was determined for each of the four climatic sections. Results of this analysis are shown in Table 36, where the storm period precipitation values have been expressed in terms of percent of annual mean precipitation at selected recurrence intervals for each section. Except for the Northwest Section, the relation between annual and storm precipitation is nearly uniform throughout the state.

The data from Table 36 and Figure 15 provide another method for obtaining isohyetal patterns. By multiplying the station annual means by the percentage value for the corresponding section and recurrence interval in Table 36, numerous points can be obtained for establishing an isohyetal map. This procedure has been illustrated for a 50-year recurrence of 1-day precipitation in Figure 16. Average sectional values from Table 36, which are also shown in Figure 16, indicate a relatively small range of values about the mean in each section. Figure 16 may be compared with Figures 2 and 3 to indicate the differences between isohyetal patterns derived from several techniques using the same statistical distribution (Frechet).

In Figure 16, the isohyetal pattern is based upon average storm frequencies for climatic sections and station annual mean precipitation. The isohyetal pattern of Figure 2 was obtained from the same sectional storm frequencies and the storm precipitation means for individual stations. Figure 3 was derived entirely from storm frequency relations for individual stations. For those who prefer an isohyetal presentation, the authors recommend the relatively simple method employed in this section, using an-

TABLE 36

RELATION BETWEEN STORM FREQUENCIES AND ANNUAL MEAN PRECIPITATION USING ANNUAL MAXIMA AND LOG GUMBEL METHOD

Storm Recurrence (Yrs.)	Percent of Annual Mean for Given Storm Period (Days) 1	2	3	5	10
		Northwest Section			
2	6.8	8.2	9.1	10.2	12.3
5	9.2	11.0	12.3	14.0	16.6
10	11.4	13.7	15.2	17.3	21.5
20	14.0	16.8	18.7	21.0	25.2
30	16.1	19.0	21.0	23.7	28.4
40	16.9	20.3	22.5	25.5	30.5
50	18.7	21.7	24.3	27.3	33.1
100	21.8	26.2	28.9	32.8	38.7
		North Central Section			
2	6.4	7.7	8.5	9.6	11.7
5	8.4	10.1	11.2	12.6	15.4
10	10.2	12.2	13.6	15.3	18.8
20	12.3	14.8	16.4	18.5	22.5
30	13.7	16.4	18.3	20.6	24.8
40	14.6	17.5	19.5	21.9	26.8
50	15.4	18.5	20.5	23.1	27.9
100	18.2	21.8	24.2	27.2	33.4
		South Central Section			
2	6.2	7.4	8.3	9.3	11.5
5	8.3	10.0	11.1	12.5	15.7
10	10.1	12.1	13.5	15.2	19.3
20	12.4	14.9	16.5	18.6	23.5
30	13.8	16.5	18.4	20.7	26.0
40	14.8	17.6	19.7	22.2	27.4
50	15.7	18.9	20.9	23.5	29.5
100	18.8	22.6	25.1	28.2	35.0
		Southeast Section			
2	6.4	7.7	8.5	9.6	11.7
5	8.4	10.1	11.2	12.6	15.3
10	10.1	12.1	13.5	15.2	18.4
20	12.0	14.4	16.0	18.0	22.0
30	13.3	16.0	17.7	19.9	24.3
40	14.2	17.1	18.9	21.3	25.9
50	14.8	17.8	19.7	22.2	26.6
100	17.7	21.2	23.6	26.6	32.6

nual station means from Figure 15 with the percentages of Table 36. For reasons given previously, the authors prefer use of sectional rather than station data for derivation of storm frequencies. The relative variability of annual precipitation is considerably less than that for storm precipitation. Consequently, the 50-year annual means for stations should provide more accurate estimates of the population annual means than do the 40-year station storm means used for estimating the population storm means from which Figure 2 is derived. There is also indication in Figure 16 that the recommended method is more statisfactory climatologically. Precipitation centers shown in extreme northwestern and southern Illinois in Figure 16 coincide with the Rock River Hills and the Shawnee Hills, the most prominent hill regions in the state,[9] which may have some augmenting effect on storm precipitation.

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